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CASE STUDY OF GEO-RISK MANAGEMENT ON TUNNELING OF HYDROPOWER DEVELOPMENT IN MYANMAR

ミャンマーの水力発電におけるトンネル施工のジオリスクマネジメントに関する研究

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ABSTRACT

Hydropower is one of indispensable power generations for a developing country because of its inexpensive production cost and environmentally benign to the nature. The scales of hydropower projects are large and construction process is complicated compared to other infrastructure projects. In parallel, it involves numerous uncertainties and risks that impact on the project achievement by cost overrunning and construction delaying. The uncertainties of on ground and underground works are generally considered to be one of the greatest sources of cost and schedule risk for the projects.

Hydropower projects need high investment cost and long construction period. Therefore, availability of enough funds, time and better construction management are vital issues to implement the projects. Time and cost are major factors for the achievement of the projects. Geo-risk management is a holistic tool for hydropower development to evaluate the risk of cost over running and construction delaying.

This research focuses on unforeseen geological risks on tunneling and how to impact the cost and schedule on construction of hydropower projects. The uncertainties of underground works, such as tunnels, are generally considered to be one of the greatest sources of cost and schedule risk for the projects. In general, tunnel excavation of hydropower projects includes those for power tunnel, diversion tunnel and access tunnel, etc. It picks up the problems and risks from the past experiences of tunneling on hydropower projects in Myanmar regarding with the geotechnical failure behavior. Comparison of some projects case study approaches is conducted in order to figure out robust results of the study.

According to case study, research findings confirm that tunneling in the region of good geology is simple and poor construction does not much effect on tunneling, but tunnel construction in poor geology faces much complicated disturbances leading to collapse and poor construction also heavily effects on tunneling. It highlights that almost projects in the weak geology of Sittaung valley area are faced with tunnel failure during construction, and it impacts on construction cost and schedule. Finally, identify and compare the geo-risk which is affected on cost over running and schedule delaying of hydropower development. Then, proposed for evaluation on geo risk reduction by improvement of human factors and

mechanical factors to improve present construction management of hydropower development in Myanmar.

And then, it points out that giving time and cost for geological investigation, and training for labors and workers who have less experience on weak geology and technical knowhow are essential requirements on tunneling of hydropower development. The research concludes that the most geo-risk behaviors on weak geology among the implementation of hydropower projects are not only depend on weak geology but also miss management on human factors.

Furthermore, it is approved that geo-risk management will not only secure the financial and schedule burden of the project but also bring good practices of preserving and enhancing project quality and social well-beings in hydropower development. For the Ministry of Electric Power, this paper discusses that to establish the research capacity building for improving on organization, procurement, finance and construction which heavily effected on tunneling of hydropower projects concerning with human factors and mechanical factors in the future. Consequently, the systematic approach adopted for geo-risk management on tunneling can minimize the cost overrun and construction delaying of the hydropower projects.

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CHAPTER 1

INTRODUCTION

1.1 Background

Unforeseeable geological risk plays an important role in the hydropower development; sometimes, it may greatly impact to the project not achieving its objectives. Hydropower project involves numerous uncertainties and risks, so construction progress rarely go according to plan. Geo-risk management is a holistic tool for hydropower development to evaluate the risk of cost over running and delaying construction schedule. However, estimation or expectation of geo-risk may not give sound decision but an aid to decision making for the project implementation.

Hydropower is one of indispensable power generations for a developing country because of its inexpensive production cost and environmentally benign to the nature. Development of hydropower project is drastically increasing in Asian countries which have drawn tremendous interest and investments in recent times, as scary of energy resources. The scales of hydropower projects are large and construction process is complicated compared to other infrastructure projects. Therefore, proper construction planning and management is essential in hydropower development.

1.2 Status of Hydropower Development in Myanmar

Myanmar is well-endowed with natural water resources because of favorable topography and tropical monsoon climate, and mostly still in virgin condition of hydropower resources. Four main rivers namely Ayeyarwaddy, Chindwin, Sittaung, Thanlwin, and theirs tributaries dominate approximately 70 percent of land surface as shown in Figure1.1. According to preliminary development plan, hydropower potential of Myanmar is estimated at about 108,000 MW. Currently Ministry of Electric Power identified hydropower potential up to 2012 indicated about 46,330 MW in total that there are as many as 92 sites each having more than 10 MW with potential total installed capacity of 46,099 MW. Similarly as many as 210 sites of small and medium size each having less than 10 MW with a total potential installed capacity of 231 MW is investigated. Regional hydropower potential of Myanmar by year 2000 is demonstrated in Figure1.2.

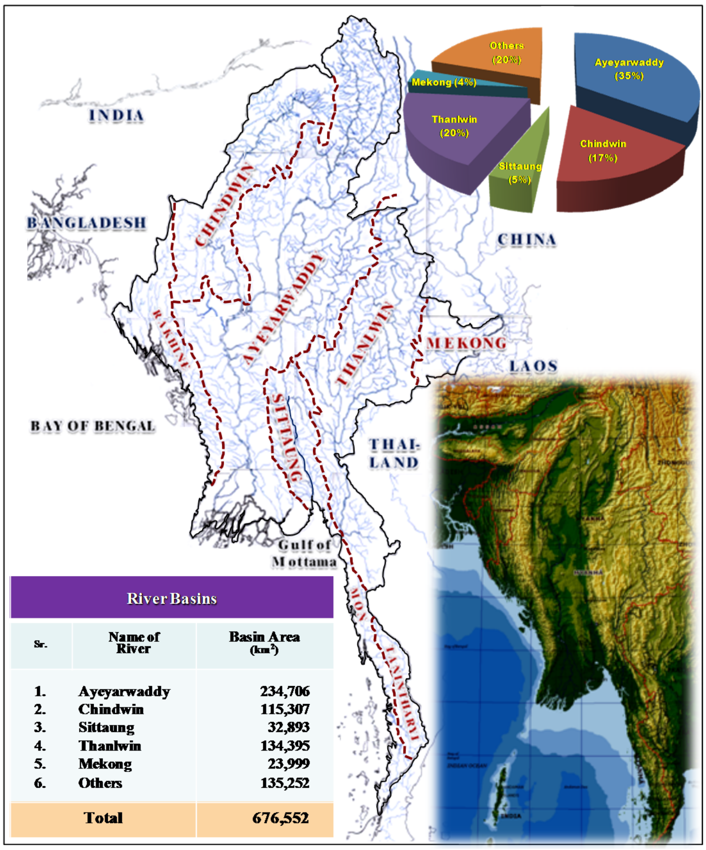


Figure 1.1: River Basin of Myanmar

Nowadays, according to Myanmar government strategies, the construction of hydropower projects is one of the priorities for development of society and economy. However, the availability of sufficient funds is a vital issue to implement the hydropower projects. At present, the required local and foreign currency is financed mainly by the Government; on the other hand, they had cooperated with foreign investors such as China, Thailand and India on joint venture basis. Besides the foreign organizations, Myanmar local companies have also taken the opportunity of investment in hydropower projects. According to September 2014 statistics, total installed capacity of Myanmar is 3970.8 MW and out of total electricity generated 74% from hydro power, 23% is from diesel and steam including natural gas, and 3% is from coal as tabulated in Table 1.1. Therefore, electricity by hydropower is majorly supplied to the country demand, and just only 6% of the country potential had already been developed and 94% of the country potential is still remaining by means of currently identified hydropower potential as tabulated in Table 1.2. Among the many ways of generation of electricity, hydropower is the most efficient way of power generation alternatives and has many favorable characteristics such as renewable, clean, reliable and flexible as illustrated in Figure 1.3.

State / Division	Number of Sites	Potential (MW)
Kachin	51	20,778
Kayah	8	954
Kayin	22	7,075
Chin	12	7
Sagaing	30	2,848
Tanintharyi	23	735
Bago	15	543
Magwe	13	370
Mandalay	14	1,424
Mon	10	303
Rakhine	14	769
Shan (East)	24	741
Shan (South)	27	7,585
Shan (Nouth)	40	5,088
Total	**303**	**49,220**

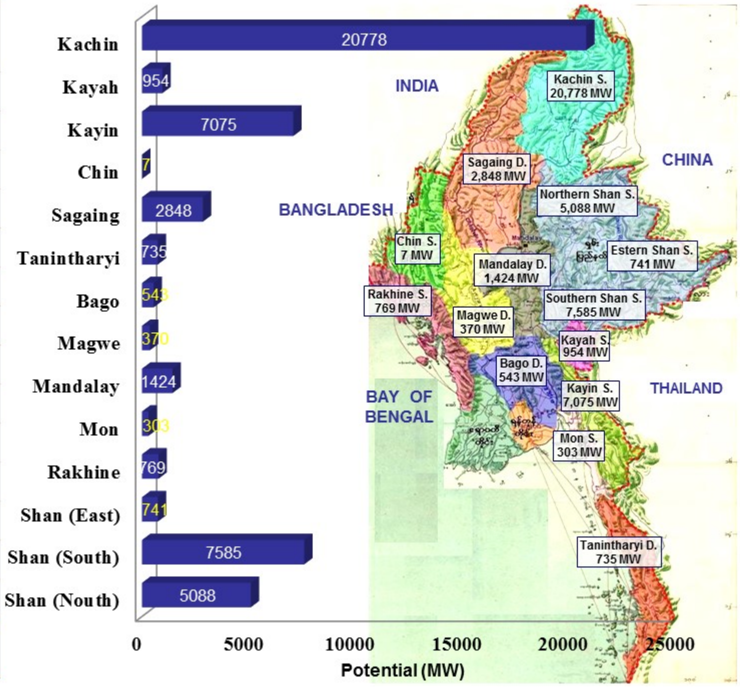
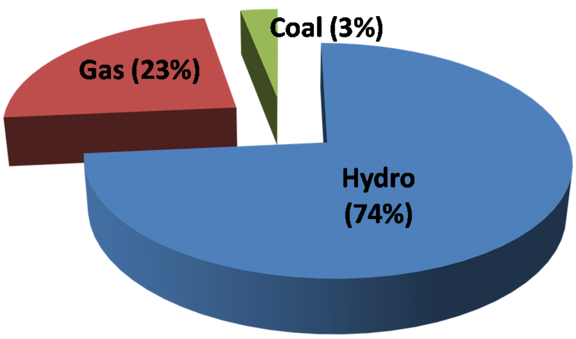


Figure 1.2: Regional Hydropower Potential of Myanmar (Year 2000)

Table 1.1: Present Electric Power Generation in Myanmar (September 2014)

Subject	Installed (MW)	Firm (MW)	Energy (GWh)
Hydropower	2,919	1,087	14,326
Coal-Based Power	120	27	600
Gas-Based (MOEP- Owned)	715.3	427	3,946
Gas-Based as BOT	216.5	215	1,457
Total	3,970.8	1,756	20,329



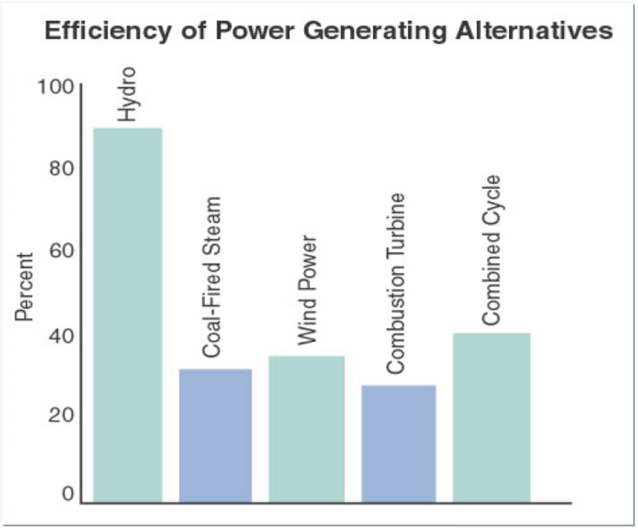


Figure 1.3: Comparison of Power Generation Efficiency (Source: USACE, 2007)

Table 1.2: Existing Large Scale Hydropower Facilities

Sr. No.	Hydropower Station	Location (State/ Region)	Installed Capacity (MW)	Annual Energy (GWh)	Operation Year
1	Baluchaung -2	Kayah State	168	1,190	1960/1974
2	Kinda	Sagaing Region	56	165	1985
3	Sedawgyi	Mandalay Region	25	134	1989
4	Baluchaung -1	Kayah State	28	200	1992
5	Zawgyi -1	Shan State	18	35	1995
6	Zawgyi -2	Shan State	12	30	1998
7	Zaungtu	Bago Region	20	76	2000
8	Thapanzeik	Sagaing Region	30	117.2	2002
9	Mone	Magwe Region	75	330	2004
10	Paunglaung	Mandalay Region	280	911	2005
11	Yenwe	Bago Region	25	123	2007
12	Kabaung	Bago Region	30	120	2008
13	Kengtawng	Shan State	54	377.6	2009
14	Shweli -1	Shan State	600	4,022	2009
15	Yeywa	Mandalay Region	790	3,550	2010
16	Dapein -1	Kachin State	240	1,065	2011
17	Shwegyin	Bago Region	75	262	2011
18	Kyeeon Kyeewa	Magwe Region	74	370	2012
19	Kun	Bago Region	60	190	2012
20	Chipwenge	Kachin State	99	599	2013
21	Thaukyegat -2	Bago Region	120	604	2013
22	Nancho	Mandalay Region	40	152	2014
23	Phyu	Bago Region	40	120	2014
	Total		**2,959**	**14,743**	

1.3 Statement of the Problem

Ministry of Electric Power (MOEP) had been trying to implement large scale hydropower projects to fulfill the electricity requirement of the country. The implementation of hydropower project involves numerous uncertainties and risks. Consequently, it impact on the project achievement by cost overrunning and construction delaying. Some geo-risks of hydropower project are listed on Table 1.3.

Table 1.3: Geo-Risk Classification on Hydropower Project

Dam & Appartenance Structures	*Feasibility Stage*	*Design Stage*	*Construction Stage*	*Operation & Maintenance Stage*
Dam	* Foundation & Abutment * Reservoir Ring	* Foundation strength * Water tightness/ seepage	* Foundation Treatment	* Excessive seepage * Pore water pressure
Intake	* Foundation level * Slope stability	* Foundation strength	* Foundation Treatment	* Landslide near by structure
Headrace Tunnel / Canal	* Alignment * Mountain geology	* Fault/ fracture on the alignment	* Groundwater	* Aging of rock & structure
Surge Tank / Head Tank	* Shaft/ tank location * Mountain geology	* Fault/ fracture on the alignment	* Groundwater	* Aging of rock & structure
Penstock	* Alignment * Foundation level	* Foundation strength	* Foundation Treatment	* Aging of foundation & structure
Powerhouse	* Foundation level * Location	* Foundation strength	* Foundation Treatment	* Landslide near by structure
Tailrace	* Foundation level * Location	* Foundation strength	* Foundation Treatment	* Pore water pressure

The uncertainties of underground works, such as tunnels, are generally considered to be one of the greatest sources of cost and schedule risk for the projects. Hydropower project needs high investment cost and long construction period. Therefore, availability of enough funds, time and better construction management are essential. Most of projects are included tunnel works. At present, 22 numbers of hydropower plants are completed and 59 projects are under planning, and construction is majorly done by joint venture with China, Thailand and India. Among the completed projects, there are 33 hydropower tunnels and total length is about 41.65 km. The following Table 1.4 and Figure 1.4 show the status of tunneling in hydropower projects of Myanmar.

Table 1.4: List of Tunneling in Hydropower Projects

Project Name	Year	Diameter/ WxH (m)	Length (km)	Acc; Length (km)
Paunglaung	1997	8.5, 10x14	4.17	4.17
Yenwe	2000	5.6	0.29	4.46
Kabaung	2000	6.5	0.28	4.73
Kun	2000	5.5	2.28	7.02
Phyu	2000	5.6	0.64	7.65
Yeywa	2000	10	0.95	8.60
Shweli	2002	8.5, 10	5.41	14.01
Upper Paunglaung	2004	10	0.30	14.31
Nancho	2005	4.7	2.24	16.54
Thahtay	2006	10	1.26	17.80
Tapein-1	2007	7, 8	0.89	18.69
Upper Keng Tawng	2008	8	0.53	19.22
Thaukyegat-2	2009	8.5, 11x13	1.07	20.29
Chipwenge	2009	5	15.53	35.83
Baluchaung-3	2009	5	4.73	40.56
Upper Yeywa	2010	10	1.09	41.65

Though tunnels of the projects in the region of hard rock are simple, the tunnels construction in poor geology face with much complicated disturbances leads to collapse, especially for Sittaung Valley Projects, which are giving many lessons for tunneling in Myanmar. Statement of geo-risk on tunneling of hydropower projects are illustrated in Figure 1.5. Therefore, this study focuses on some projects for making comparison which is implemented by different parties and located in the complex geological area known as Sittaung Valley Projects.

In the Sittaung valley river basin, four projects are located on west to Sittaung river and six projects are located on east to Sittaung river as shown in Figure 1.6. Among the Sittaung valley projects, the upper most three projects are located in very good geological area, and middle and lower other seven projects are located in very weak geological area which are having engineering challenges of different tunneling methods in different geological conditions of rocks and solving the problems daily encountered during construction. Among these, four projects are selected for case study of tunneling practices which are having similar tunnel structures and different geological conditions. Comparison of some project case studies is conducted in order to figure out robust results of the study.

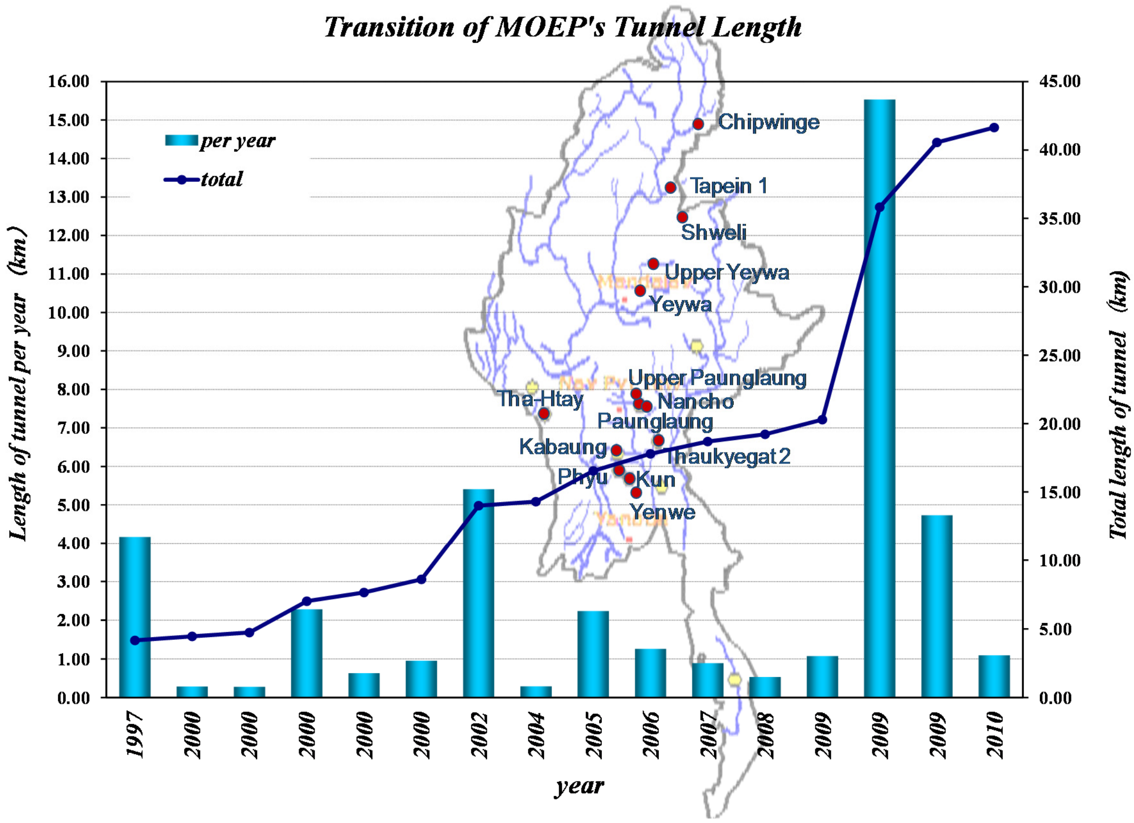


Figure 1.4: Progress of Tunneling in Hydropower Projects

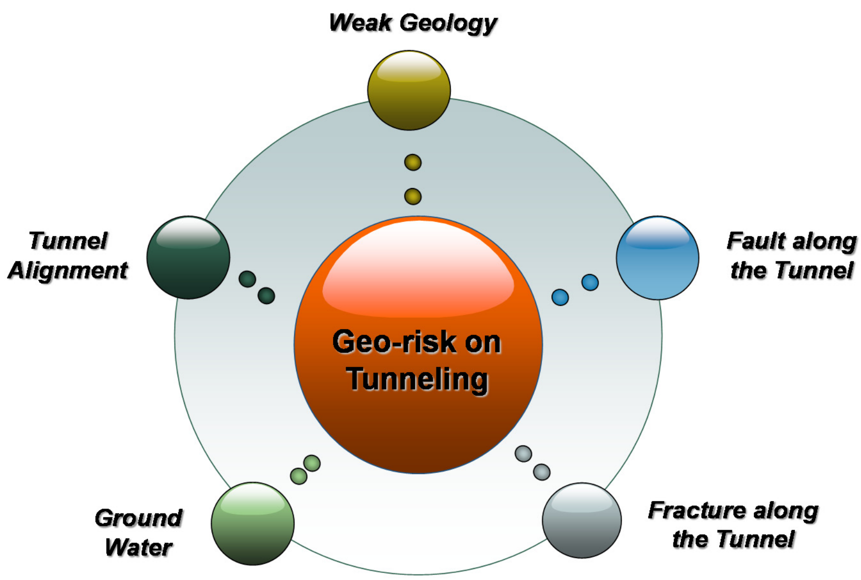


Figure 1.5: Statement of Geo-Risk on Hydropower Tunnels

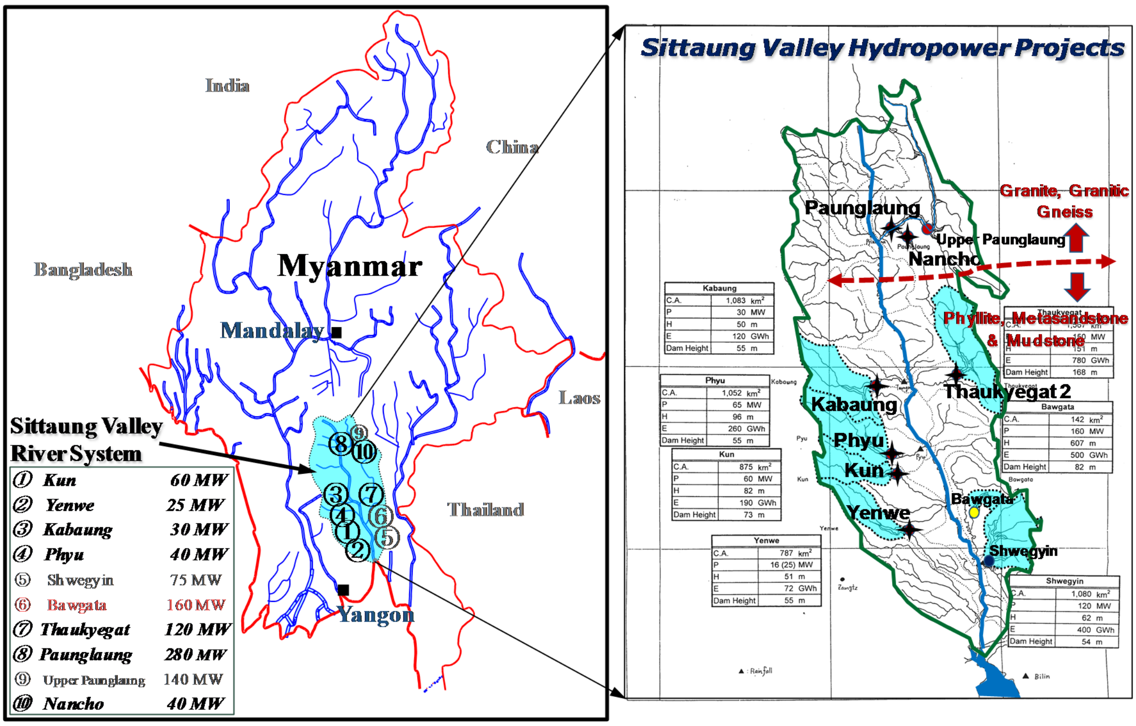


Figure 1.6: Location Map of Sittaung Valley River Basin Hydropower Projects (Source: KEPCO, 2008)

1.4 Objectives of the Study

The main objective of the study is to focus on unforeseen geological risks of tunneling and how to impact the cost and schedule on construction of hydropower projects. Time and cost are major factors for the achievement of the projects. It picks up the problems and risks from the past experiences of tunneling on hydropower projects in Myanmar regarding with the geotechnical failure behavior.

The specific purposes of the study are as follows:

1. To apply risk assessment and response on hydropower development.
2. To identify the geo-risk factors and impacts on cost and schedule of tunneling among the hydropower projects.
3. To compare the rational of essential geo-risk reduction factors on tunneling which are found in weak and good geological area.
4. To clarify the potential risks and evaluation on risk factors by means of human factors and mechanical factors.

5. To approve the geo-risk factors and quantify the effectiveness of ideal risk responses of construction management on Hydropower Development.

1.5 Scope of the Study

The overall framework and scope of research study is based on the geo-risk evaluation of tunnel excavation on hydropower projects, which has experienced from Myanmar. The dissertation which consists of six chapters is introduced as follows:

Chapter 1: Gives background, objectives and scope of introduction of this research.

Chapter 2: Reviews the literatures related to tunneling of hydropower projects.

Chapter 3: Presents the research methodology for tunneling practice.

Chapter 4: Compares and evaluates on geo-risk of past tunneling of Sittaung Valley Hydropower Projects.

Chapter 5: Accesses the geo-risk management on tunneling of hydropower projects.

Chapter 6: Gives conclusion of this research study and recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 Geo-Risk in Hydropower Projects

2.1.1 Introduction

Assessment of hydropower projects involves numerous uncertainties that affect the overall feasibility of a project. A comprehensive feasibility evaluation will include hydrological, geological, engineering design, construction, commercial, environmental, and social studies. Among these geology is one of the greatest source of uncertainties for the hydropower projects. Geologic uncertainties from investigations in the vicinity of above ground structures. This will affect foundation levels, slope stability, foundation design parameters, and ultimately the quantities and cost of the work. Geologic uncertainties for underground structures (i.e. tunnels and powerhouse). Geology is commonly characterized according to a rock class that designates the likely over break and costs for rock support and lining. The uncertainty associated with geologic conditions and the associated impacts on construction productivity is significant (Peter J. Rae).

Flanagan and Norman (1993) defined that risk can manifest itself in numerous ways, varying over time and across activities. Especially, it stems from uncertainty, which in turn is caused by a lack of information.

Uncertainty:

The lack of certainty, a state of having limited knowledge where it is impossible to exactly describe the existing state, a future outcome, or more than one possible outcome (En.wikipedia.org).

Risk:

A state of uncertainty where some possible outcomes have an undesired effect or significant loss (En.wikipedia.org).

"Risk" in the discipline of engineering is generally defined as an expected loss incurred by an event that may cause damage (Ohtsu, 2012).

Miller and Lessard (2001) suggested that risk in a construction project is unavoidable and affects productivity, performance, quality and budget significantly by considering the complex, dynamic and challenging nature of construction projects, however, risk can be minimized, controlled, transferred, shared or insured.

Risk identification is a repetitive process because new risks may become known as the project progresses through its life cycle and previously-identified risks may drop out. Risk response planning is the process of developing options, and determining actions to enhance opportunities and reduce threats to the project's objectives (Project risk management hand book, 2007).

2.1.2 Risk Management

Chapman and Ward (1997) defined and pointed out that risk is present in every aspect of our lives: thus risk management is universal but in most circumstances an unstructured activity, based on common sense, relevant knowledge, experience and instinct. All projects involve risk-the zero risk project is not worth pursuing. Various parties' involvement and relationships bring fundamental complications which have a profound influence on project uncertainty and project risk within a project. A broad definition of project risk is "the implications of the existence of significant uncertainty about the level of project performance achievable". A source of risk is any factor that can affect project performance, and risk arises when this effect is both uncertain and significant in its impact on project performance. Setting tight cost and time targets makes a project more cost or time risky by definition, since achievement of targets is more uncertain if targets are "tight". However, inappropriate targets are themselves a source of risk, and a failure to acknowledge the need for a minimum level of performance against certain criteria automatically generates risk on those dimensions.

Risk management is defined as a systematic controlling procedure for predicted risks to be faced in an investment or project of organization. Risk management is the systematic process of planning for, identifying, analyzing, responding to, and monitoring project risk. Risk management assists in setting priorities, allocation resources and implementing actions and processes that reduce the risk of the project not achieving its objectives (Project Risk Management Handbook, 2007).

Flanagan and Norman (1993) proposed a risk management system must be practical, realistic and must be cost effective. Risk management need not be complicated nor require the

collection of vast amounts of data. It is a matter of common sense, analysis, judgment, intuition, experience, gut feel and a willingness to operate a disciplined approach to one of the most critical features of any business or project in which risk is generated. Naturally the risk management system must be applied to each option under consideration. Generally, the stages are:

- Risk identification: Identify the source and type of risks
- Risk classification: Consider the type of risk and its effect on the person or organization
- Risk Assessment: Evaluate the consequences associated with the type of risk, or combination of risks, by using analytical techniques
- Risk response: Consider how the risk should be managed by either transferring it to another party or retaining it.

2.1.3 Geo-Risk in Underground Structures of Hydropower Projects

Hydropower construction has many unknowns and things rarely go according to plan. Estimation or expectation of geo-risk will not provide the company or the investors with sufficient information on which to base a sound decision. Techniques for quantifying risk as an aid to decision making (Flanagan and Norman, 1993). To deal with uncertainty of underground geological conditions in an actual construction project involving underground work, geological investigations and their result are used in the subsequent design phase. It should be noted, however, that there are always uncertainties in geological data obtained prior to the design phase, as it is practically impossible to investigate the geology of the whole construction site in preliminary investigations. Based on the schematic drawing shown in Figure 2.1, the level of uncertainty of a construction project involving underground work can be explained as follows: The level of uncertainty of geological conditions becomes the highest at the post preliminary investigation feasibility study phase, which is lowered at detail design phase and further at the construction phase, as more geological information becomes available. Also as shown in Figure 2.1, it goes without saying that the extent of geological risk changes (referred to as “geological investigation uncertainty level” in the figure) is affected by the quality and quantity of in-situ investigations performed (Ohtsu et al., 2004).

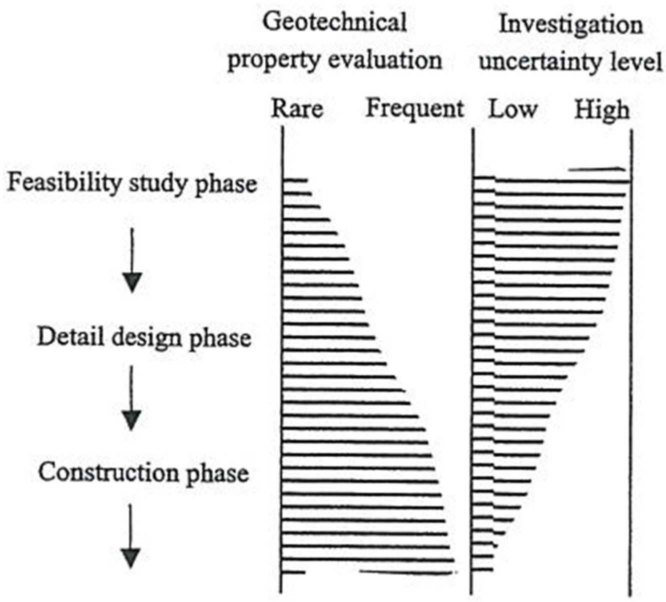


Figure 2.1: Uncertainties Involved in Risk Factor Evaluation, Geotechnical Property Evaluation and In-situ Investigation (Source: ISRM, 2004)

2.2 Features of Hydropower Project

2.2.1 Introduction

The terminology of hydropower project is output power, discharge, head and annual power generation. To get the most power generation, it requires getting maximum plant discharge and most effective head. The waterway plays an essential role for the hydropower projects. There are two types of waterway such as pressure type and non-pressure type. It consists of Intake, Settling Basin, Headrace Tunnel/ Canal, Surge Tank/ Head Tank, Penstock, Powerhouse and Outlet. Following Figure 2.2 shows major structures of a hydropower project.

2.2.2 Types of Hydropower Project

Conventionally, hydropower generation can be classified from a viewpoint of "control of the river flow" and "method of head acquisition". From a view point of control of the river flow, generation type can be divided into three types, (1) run-of-river type, (2) pondage type and (3) reservoir type as illustrated in Figure 2.3. From a viewpoint of method of head acquisition, generation type can be also divided into three types, (1) waterway type, (2) dam type, and (3) dam and waterway type as illustrated in Figure 2.4 (NEF, 1996).

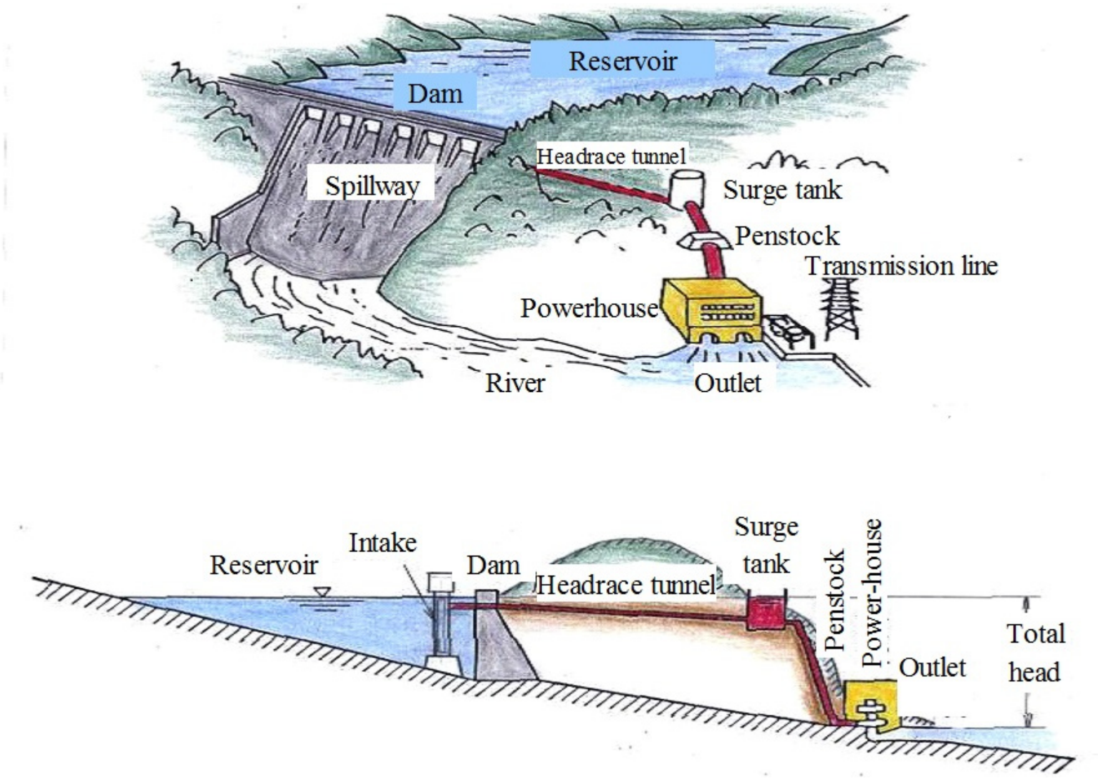


Figure 2.2: Major Structures of a Hydropower Project (Source: JEPIC, 2008)

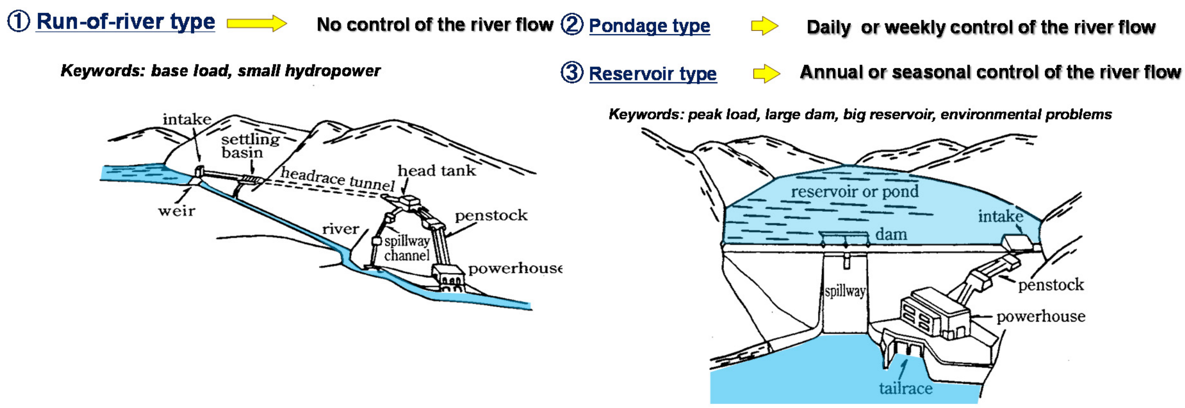


Figure 2.3: From a Viewpoint of Control of the River Flow (Source: KEPCO, 2005)

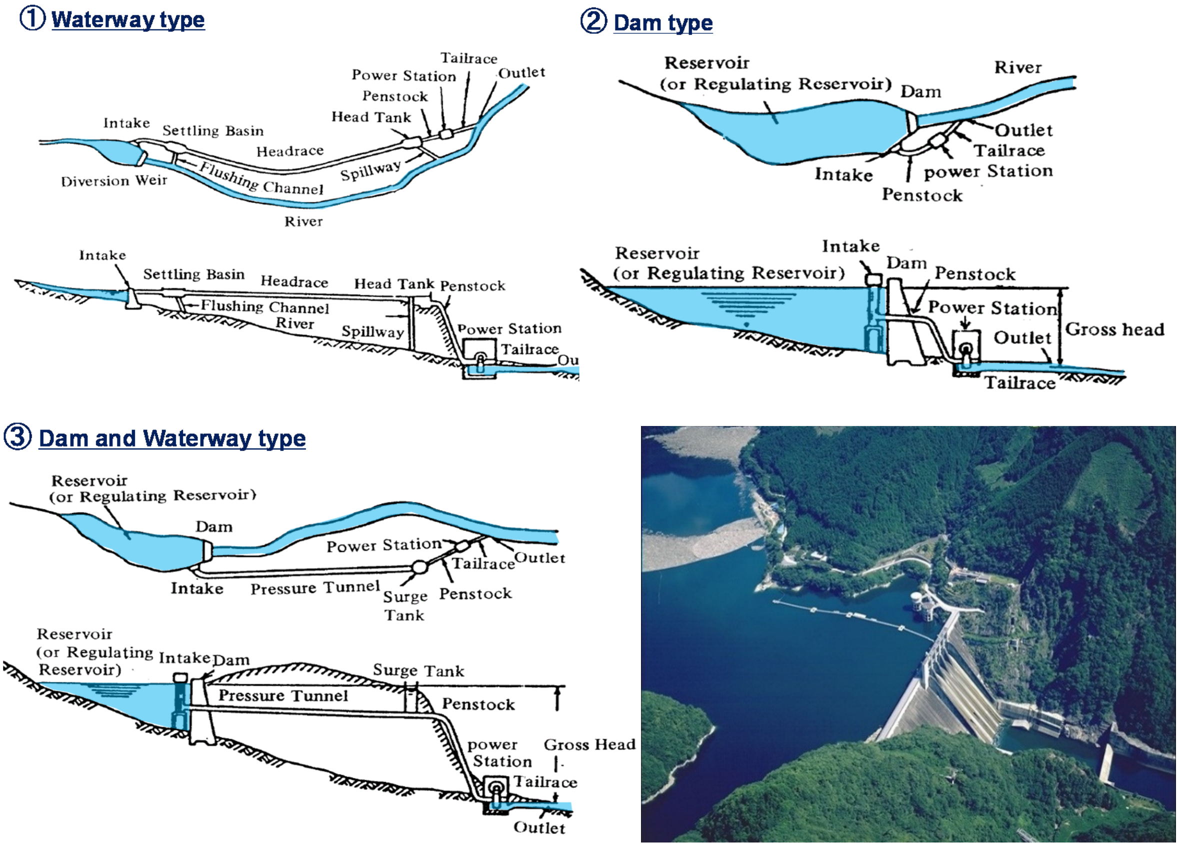


Figure 2.4: From a Viewpoint of Method of Head Acquisition (Source: KEPCO, 2005)

2.2.3 Tunnels in Hydropower Project

In the hydropower project, dam and waterway hydraulic structures are main components for power generation and they are conveying water efficiently from reservoir to tailrace. In general, tunnel excavation of hydropower projects includes those for headrace tunnel, diversion tunnel and access tunnel, etc. It plays an essential role for the hydropower projects. For the construction of dam, diversion tunnel or conduit is vital structure. For the power portion, waterway which consists of Intake, Headrace Tunnel/ Canal, Surge Tank/ Head Tank, Penstock, Powerhouse and Tailrace is essential structures. In the waterway structure, headrace tunnel is dominant and major structure from the viewpoints of safety, economic and environmental issue.

Waterway may vary according to type of hydropower generation. In the waterway type, head is mainly acquired by the gap in elevation between the entrance and the exit of waterways. In the dam type, head is mainly acquired by the height of dam and a power house is installed near the dam. In the dam and waterway type, it is a combination of the two types mentioned above, waterway type and dam type. Tunnel/ canal works from the inlet to the head tank or surge tank is referred to as the headrace (NEF, 1996).

According to these several types of hydropower projects, have to make a plan and design for each waterway structures based on the feasible topography condition. The dam type does not have headrace structure. Consequently, it is not necessary to consider for waterway alignment. However, waterway type and dam and waterway type have to be considered not only waterway alignment but also waterway structures setting to get the most effective power generation. The dam and waterway type development was normally chosen to increase both installed and firm power at the reasonable sacrifice of the economy.

2.2.4 Technical Considerations on Waterway

Generally, in a waterway type, and dam and waterway type power plant, it is not unusual for the construction costs of the headrace to exceed half of the construction costs. Thus, selection and design of the tunnel/ canal route should be carried out with due care. Where solid foundation can be attained with soil depth of more than (3) times the tunnel diameter up to the ground surface, it is more advantageous to have a tunnel rather than an open canal. Following Figure 2.5 shows the technical considerations on waterway of hydropower project (KEPCO, 2005).

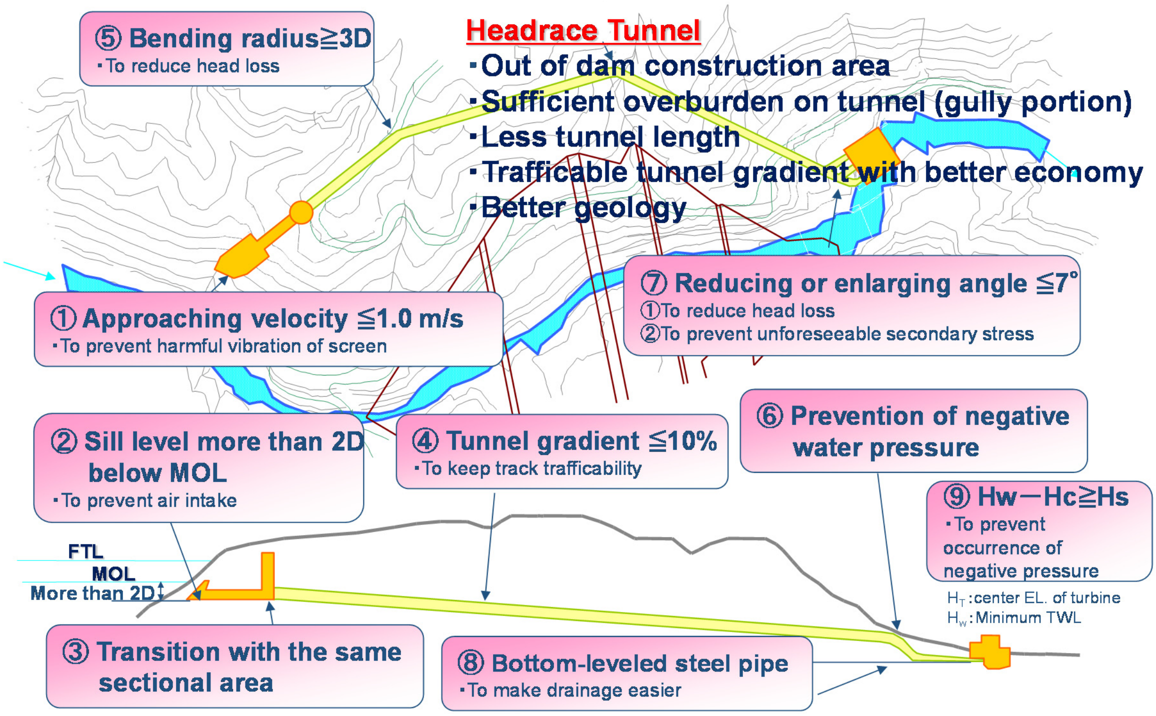


Figure 2.5: Technical Considerations on Waterway of Hydropower Project

(Source: KEPCO, 2005)

In the NEF (1996) and KEPCO (2008) guided that the design criterion in considering the alignment of headrace tunnel are sufficient in thickness of overburden in the reach of gully portions, shorter in tunnel length and better in geological condition. Generally, lining is provided for tunnel as the following reasons:

- To reduce energy loss due to friction of flowing water
- To provide strength against earth pressure from outside and water pressure inside
- To reduce water leaking.

There are two types of headrace tunnel such as pressure type and non-pressure type.

Non pressure tunnel means free-flow tunnel. As a free-flow tunnel cross section, a horseshoe-shape is generally used. This is due to fact that it is easy to build, strong to external forces, and economically efficient. When uniform water flows, the hydraulic gradient of Tunnel directly affects the cross section, and as the hydraulic gradient is larger, the cross sectional area can be smaller, reducing the construction cost. However, due to large head loss, greater power loss entails. Therefore, economically proper cross section and gradient must be decided in the design. In the case of non-pressure conduit, the gradient shall be generally designed as 1/1000~ 1/1500. The lining of non-pressure tunnel must ensure security against earth pressure.

Most of the pressure tunnels are round shaped. In case the rock bed is in good condition, for easier construction, almost round-shaped standard horseshoe shape is adopted. The flow rate inside the pressure tunnel is directly related to hydraulic gradient regardless of the tunnel gradient. The smaller cross section of pressure tunnel is, the lower the construction cost but the output and generated energy decreases as head loss increases. The larger the tunnel cross section is, the higher the cost but the output and generated energy increases as well. A few patterns of tunnel cross sections are prepared for selection the most cost effective alternative. In the case of pressure tunnels, reinforced concrete is used for large pressure locations, and at time steel plates are used as lining. As for the design load, the internal pressures are hydrostatic pressure, water hammer pressure, and surging pressure, etc. Among the external pressures which include outer water pressure, grouting pressure, and rock pressure, grouting pressure is usually dominant.

On both pressure and non-pressure tunnel, it needs special take care for portal area inlet or outlet where thin overburden place loosened by excavation. It is simple that thinner arch action is less stable than bigger arch action as shown in Figure 2.6. This condition is more serious for pressure tunnel of reservoir type. There is a possibility that surrounding rock will be loosened under the situation of initial state of reservoir impounding especially weak geological places. Reason why, the uncertainties of underground works, such as tunnels, are generally considered to be one of the greatest sources of cost and schedule risk for the hydropower projects.

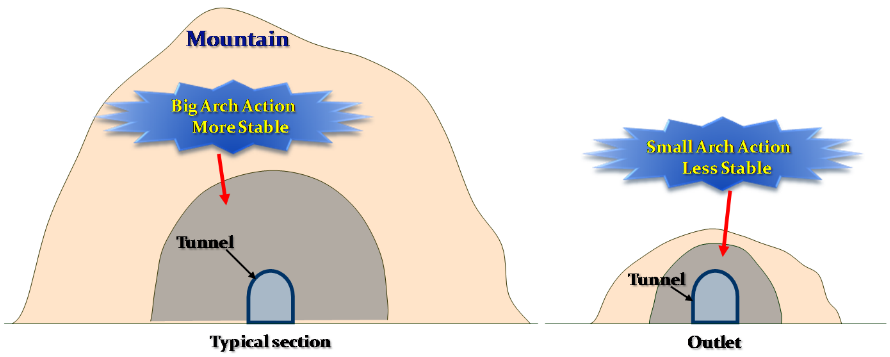


Figure 2.6: Arch Action for Inner and Outer Area of Tunnel

2.3 Tunneling Practices on Hydropower Projects

2.3.1 Introduction

In recent years, there have been significant engineering developments in tunneling technology with the experience of major international infrastructure projects such as rail and metro systems as well as road tunnels in Cities. Nowadays, tunnels are one of the major transportation systems in the world. According to population growth, economic and social requirements, tunnel structure is highly recommendable and increase demand with various purposes for worth living and a high performance. Among the tunneling development, hydropower tunnels are also one of the essential structures for power production and greatest sources in the country. As for Japan, many kinds of tunnel structures are developed such as road, railway, hydropower, electric power, telephone etc. and the total length of tunnel reaches about 15,330 km as shown in Figure 2.7.

In Japan, there is ten numbers of large whole sale electric power companies which were founded after the Second World War and monopolized power supply in their respective service areas as shown in Figure 2.8. Their name was given by the representative city or the traditional regional names of their service areas. Among these ten companies, Kansai

Electric Power Co., (KEPCO) is the second largest private power company in Japan and most of power plants are hydropower plants which has 145 hydropower, 18 thermal power and 3 nuclear power plants. Most of hydropower plants are included tunnel structures and the total length of tunnel reaches about 410 km as illustrated in Figure 2.10 (KEPCO, 2005).

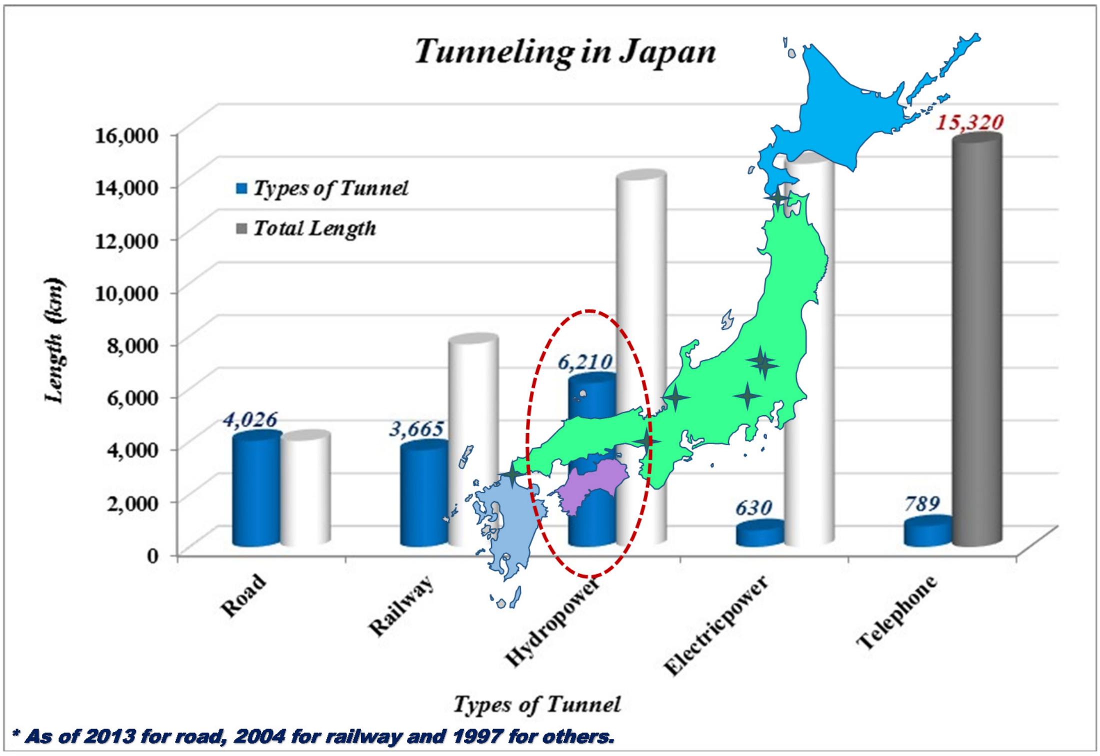


Figure 2.7: Tunnel Development in Japan (Source: MLITT, 2013)

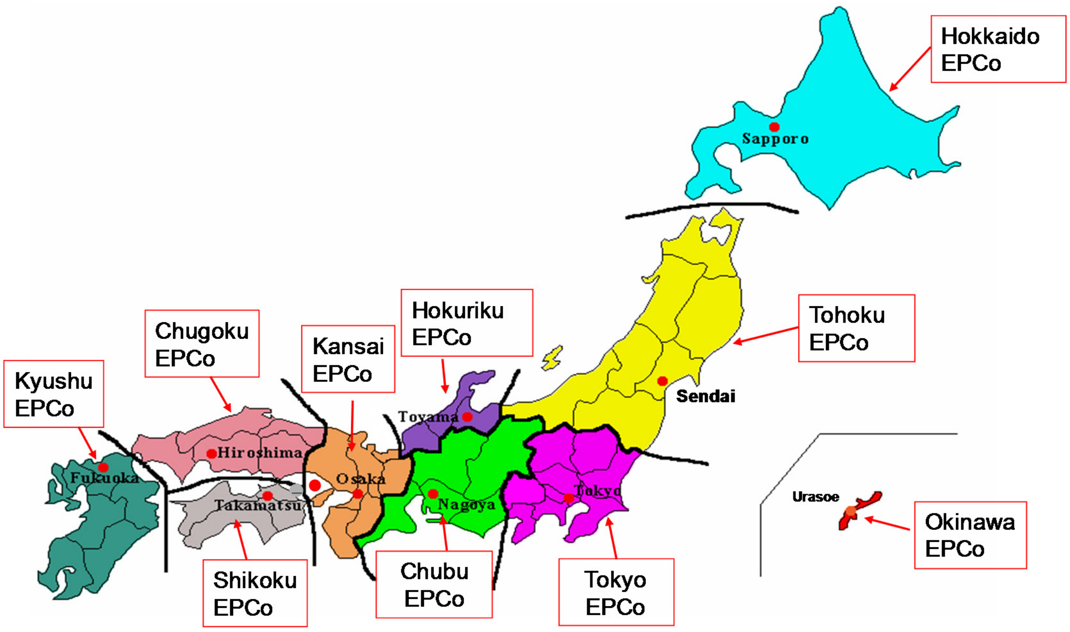


Figure 2.8: Ten Electric Power Companies in Japan (Source: JEPIC, 2008)

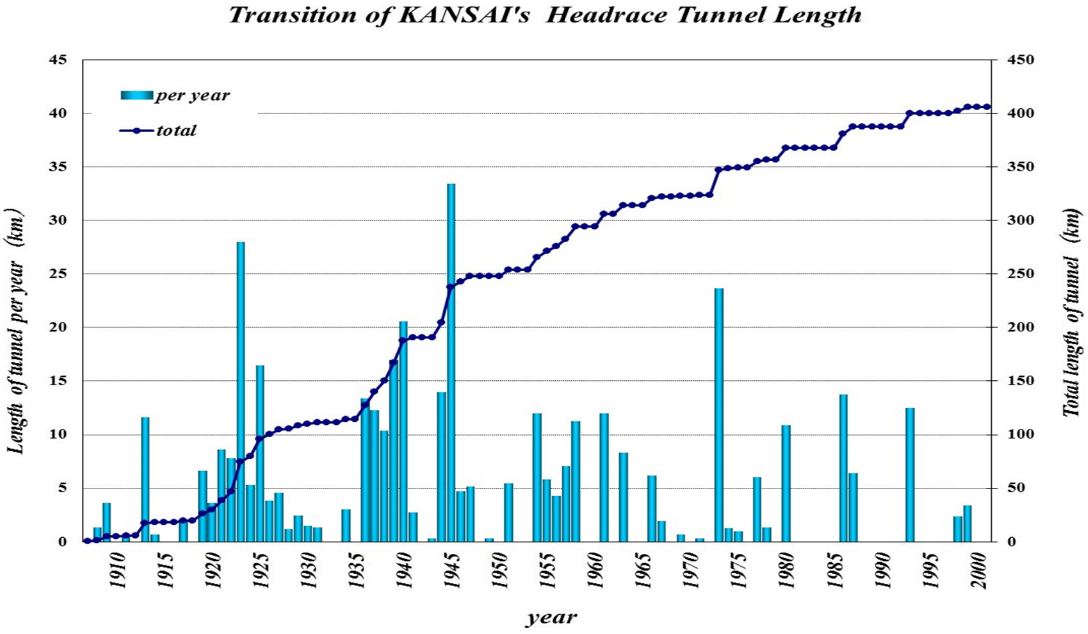


Figure 2.9: Development of Hydropower Tunnels by KEPCO

(Source: KEPCO, 2001)

2.3.2 Background of Tunneling in Myanmar

Myanmar is second largest in territorial size among the ASEAN countries and majorly central low land is widely spread with high population rather than North, East and Western mountainous area. Therefore, highway access with tunneling is rarely to see except railway tunnel. Railway tunnels were developed since pre-war and total length is about 2.6 km with 12 tunnels until 2013. Another major tunnel developer is Ministry of Electric Power (MOEP) which had been trying to implement large scale hydropower projects to fulfill the electricity requirement of the country. Most of hydropower projects are included tunnel structures. As aforesaid in Chapter 1, hydropower tunnels total length is about 41.65 km as shown in Figure 2.10.

As mentioned previously, Table 1.3 and Figure 1.4, show the status of tunneling in hydropower projects of Myanmar. Among the Sittaung Valley Projects, some projects were located in excellent geological area but most of other projects are located in weak and complex geological area which are giving many lessons for tunneling in Myanmar. Therefore, this study focuses on some projects for making comparison which is located in the different geological area and implemented by different parties.

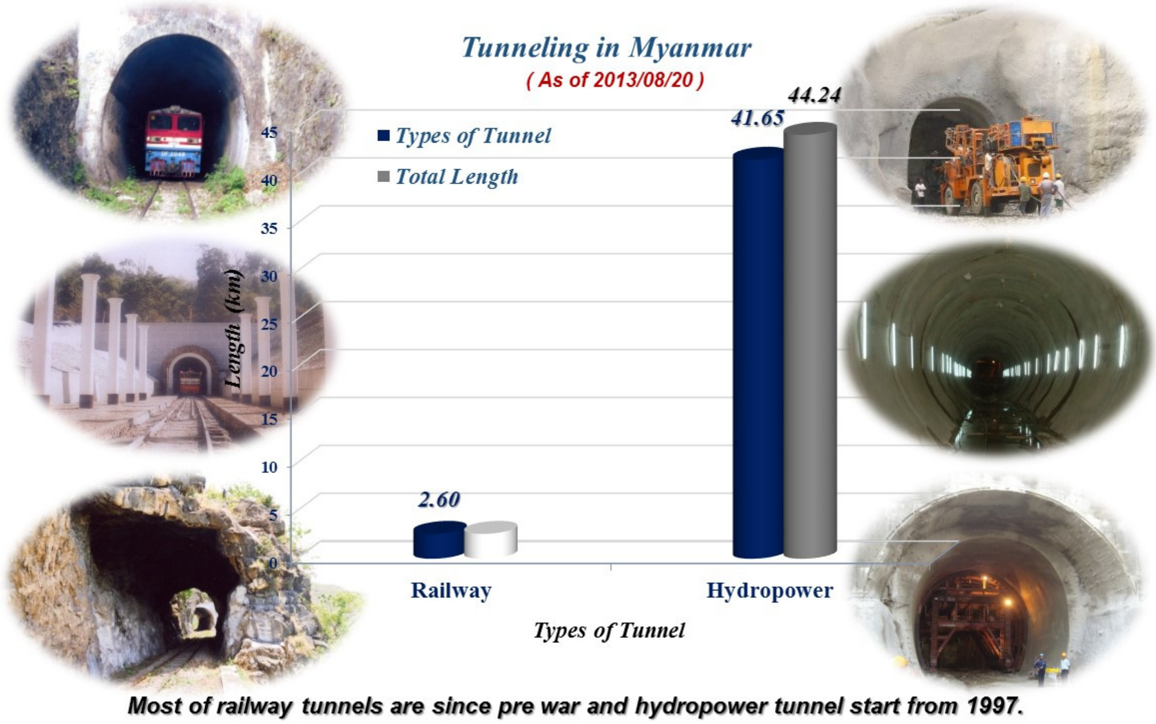


Figure 2.10: Tunnel Development in Myanmar

2.3.3 Tunneling Methods

Most of developing countries are still using traditional tunnel driving methods including Myanmar except mechanized tunneling technology such as Tunnel Boring Machine (TBM). Tunneling technology made a major advance with the application of tunnel boring machines which are capable of excavating large-diameter tunnels and installing lining and supports much quicker and more precisely than traditional methods. Tunneling has always been and will continue to be an engineering activity which is associated with uncertainty and with consequent risk of cost over-runs. There are no simple answers to these problems since, however thorough a site investigation, the rock ahead of a tunnel is unknown until it has been exposed in the face. To perform efficient tunnel works, suitable tunnel method and equipment should be chosen depending on the geology encountered during tunnel excavation. Before starting tunnel excavation, geological surveys along a tunnel alignment should be carried out to obtain the geology information, even though it is difficult to foresee the geology ahead of the tunnel excavation. According to the geological condition, tunnel driving methods are selected as illustrated in Figure 2.11.

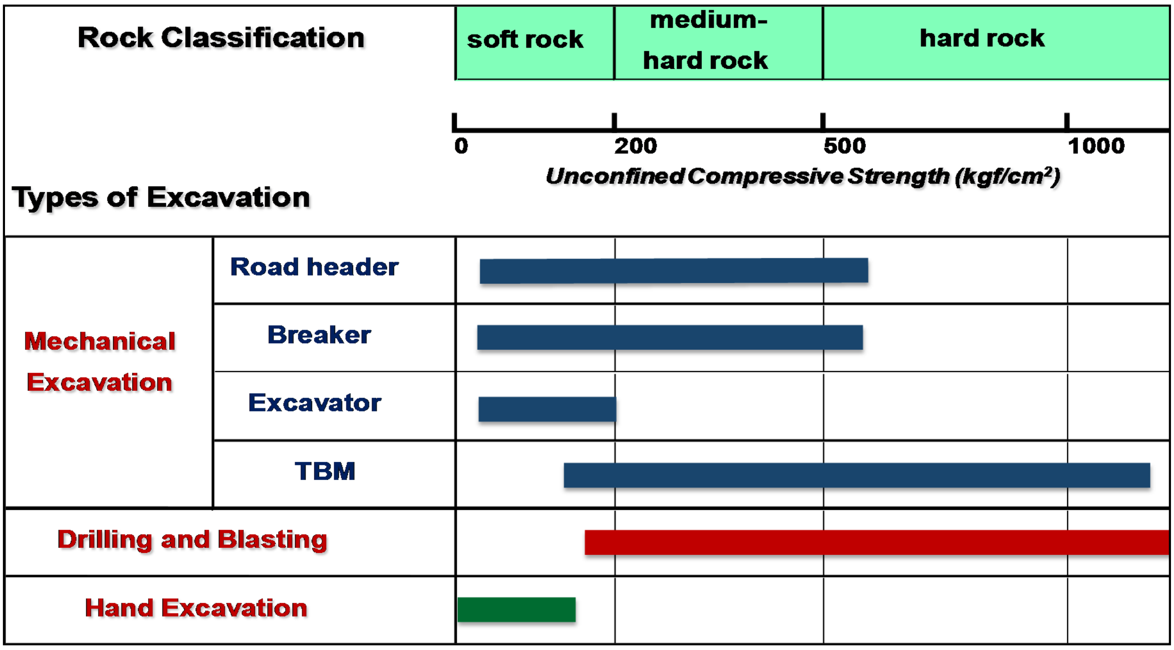


Figure 2.11: Tunnel Driving Method by Rock Classification (Source: KEPCO, 2001)

There are two types of tunneling methods widely used in hydropower projects. Those are Conventional Method and New Austrian Tunneling Method (NATM) as illustrated in Figure 2.12. While the prior one is implemented by using steel ribs, lagging and blocking, the next one preserve the strength of surrounding ground with flexible support systems such as shotcrete and rock bolts are efficiently applied based on elastic deformation in ground, but exclude loosening it. The Conventional Method has been a basic and common tunnel excavation method and it is recommendable for the tunnel excavation under the poor situation. While a tunnel is being excavated, stabilizing the ground by steel ribs with lags and blocks is the theory of conventional method. The steel ribs will support loosen ground especially at crown portion. Even in the case of tunnel excavation by NATM, the Conventional Method is often adopted in poor geological zones and fractured fault zones along tunnel. Shotcrete work is the most important in the theory of NATM and no shotcrete means any support on NATM (KEPCO, 2008).

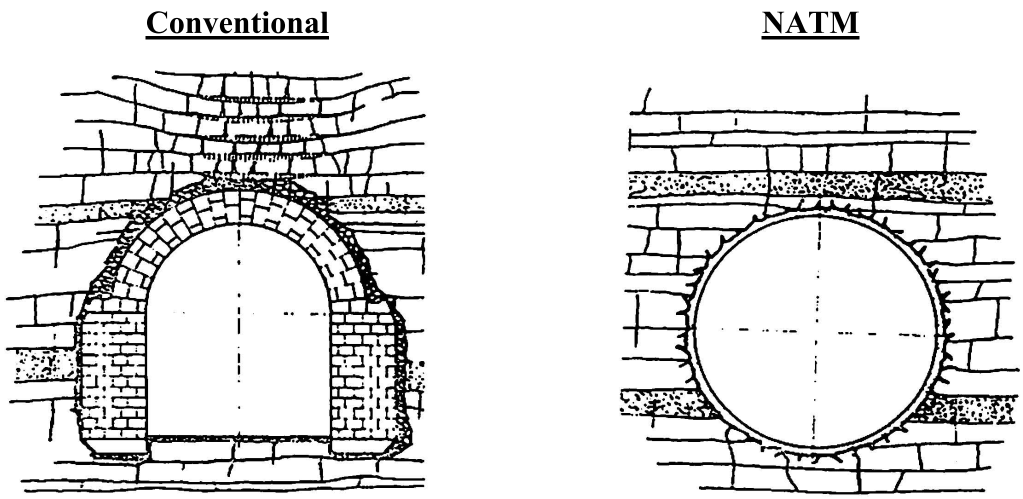


Figure 2.12: Tunnel Structures by Conventional and NATM (Source: KEPCO, 2008)

New Austrian Tunneling Method (NATM), Rabcewicz (1948) presented the basic principles of the concept were formulated and the essence was "with a flexible primary support a new equilibrium shall be reached. This shall be controlled by in-situ deformation measurements. After this new equilibrium is reached an inner arch shall be built. In specific cases the inner arch can be omitted". The main principles of NATM are:

- The main load-bearing component of the tunnel is the surrounding rock mass. Support is 'informal' i.e. it consists of earth/rock-anchors and shotcrete, but support and final lining have confining function only.

- <u>Maintain strength of the rock mass</u> and avoid detrimental loosening by careful excavation and by immediate application of support and strengthening means. Shotcrete and rock bolts applied close to the excavation face help to maintain the integrity of the rock mass.

- <u>Rounded tunnel shape</u>: avoid stress concentrations in corners where progressive failure mechanisms start.

- <u>Flexible thin lining</u>: The primary support shall be thin-walled in order to minimise bending moments and to facilitate the stress rearrangement process without exposing the lining to unfavourable sectional forces. Additional support requirement shall not be added by increasing lining thickness but by bolting. The lining shall be in full contact with the exposed rock. Shotcrete fulfils this requirement.

- Statically the tunnel is considered as a thick-walled tube consisting of the rock and lining. The closing of the ring is therefore important, i.e. the total periphery including the invert must be applied with shotcrete.

- <u>In situ measurements</u>: Observation of tunnel behaviour during construction is an integral part of NATM. With the monitoring and interpretation of deformations, strains and stresses, it is possible to optimise working procedures and support requirements.

The concept of NATM is to control deformations and stress rearrangement process in order to obtain a required safety level. The NATM method is universal, but particularly suitable for irregular shapes. The simplified steps of an underground transition created with NATM are shown below.

Step 1: Cutting a length of tunnel, here with a roadheader

Step 2: Applying layer of shotcrete on reinforcement mesh

Step 3: Primary lining applied to whole cavity, which remains under observation

Step 4: Final lining applied. Running tunnels continued

Figure 2.13: Simplified Steps of an Underground Transition Created with NATM (Source: Rabcewicz, 2004)

The following diagrams show the flow chart of tunnel excavation work by the Conventional Method and New Austrian Tunneling Method (NATM) which is widely adopted in Sittaung valley hydropower projects. Considering the workability and quality of tunnel work under

normal geological condition, NATM is generally superior to the Conventional Method (KEPCO, 2005).

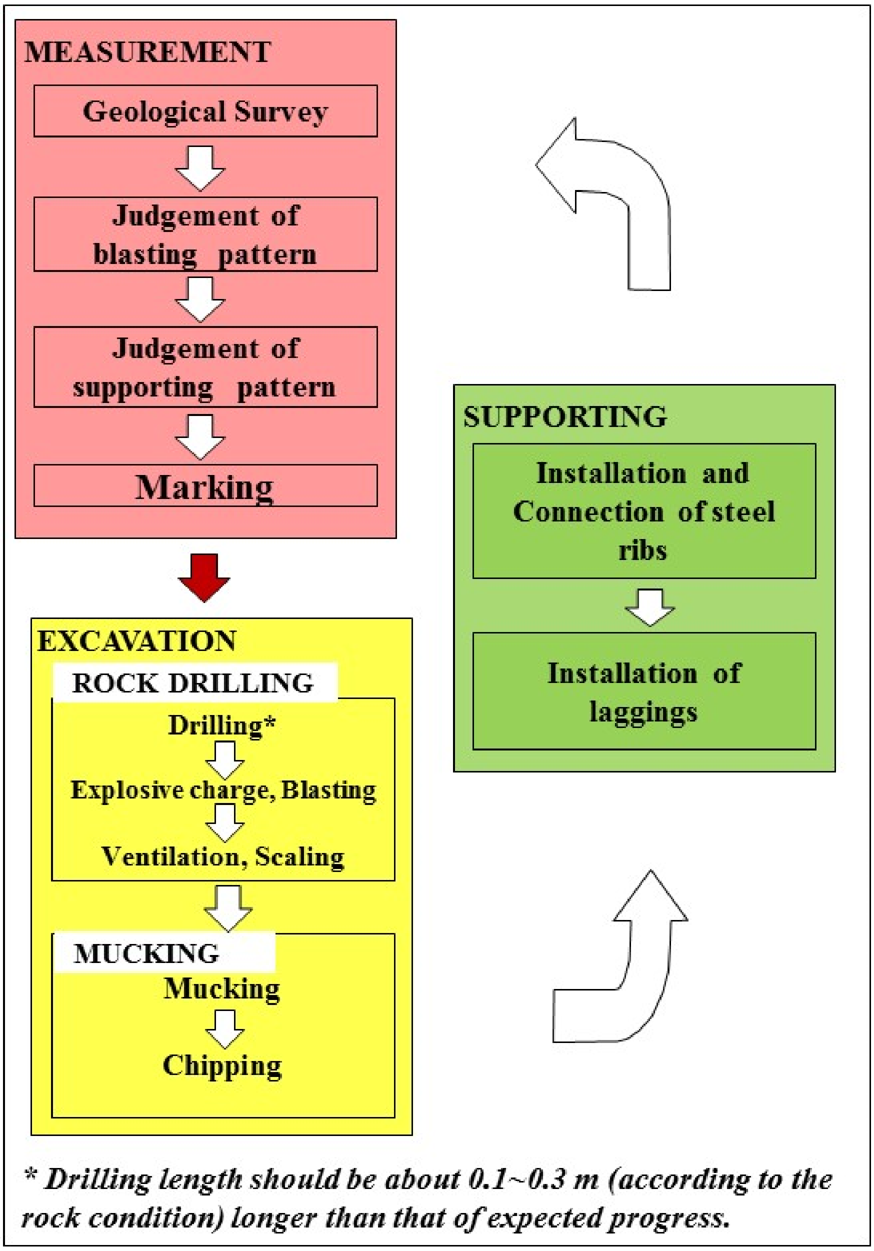


Figure 2.14: Flow Chart of Conventional Method of Tunneling

(Source: KEPCO, 2005)

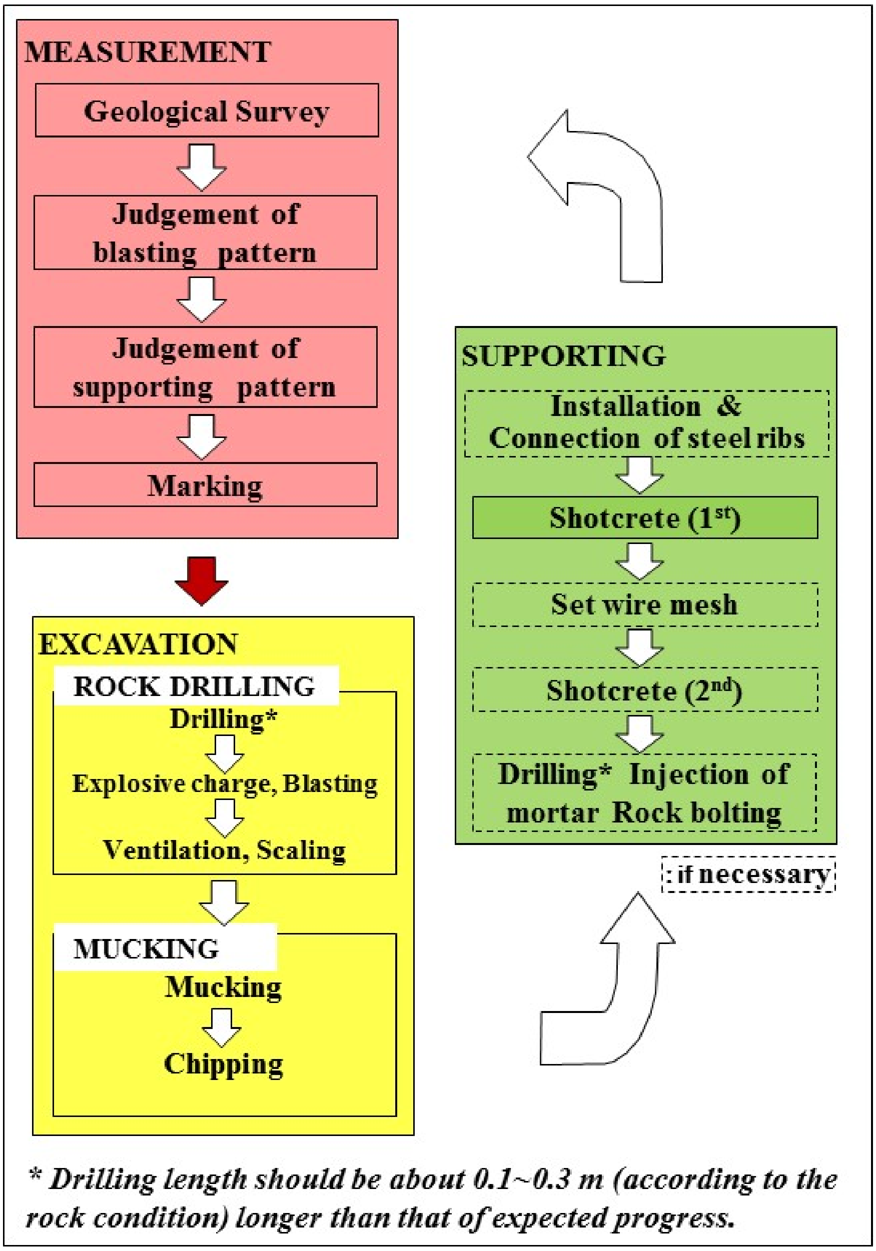
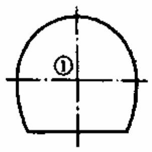
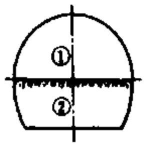
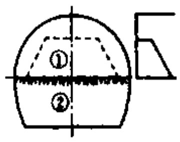


Figure 2.15: Flow Chart of NATM Method of Tunneling (Source: KEPCO, 2005)

As to tunnel excavation, it is essential to stabilize loosen excavated rocks with supports such as shotcrete, steel ribs, rock bolts as immediately as possible. Pressure of loosened ground is caused by the ground loosened by excavation and acts directly on the supports and linings. In the case of hard and medium hard ground developing cracks, it is required to implement supporting works promptly in order to restrain deformation of excavated face and behavior of ground at beddings and joints. In simple cases in which the geology and rock characteristics are predictable, the engineer may specify a pattern of rock bolts with mesh and shotcrete as required. The following Table 2.1 shows the excavation methods, and their advantage and disadvantage of tunneling (KEPCO, 2008).

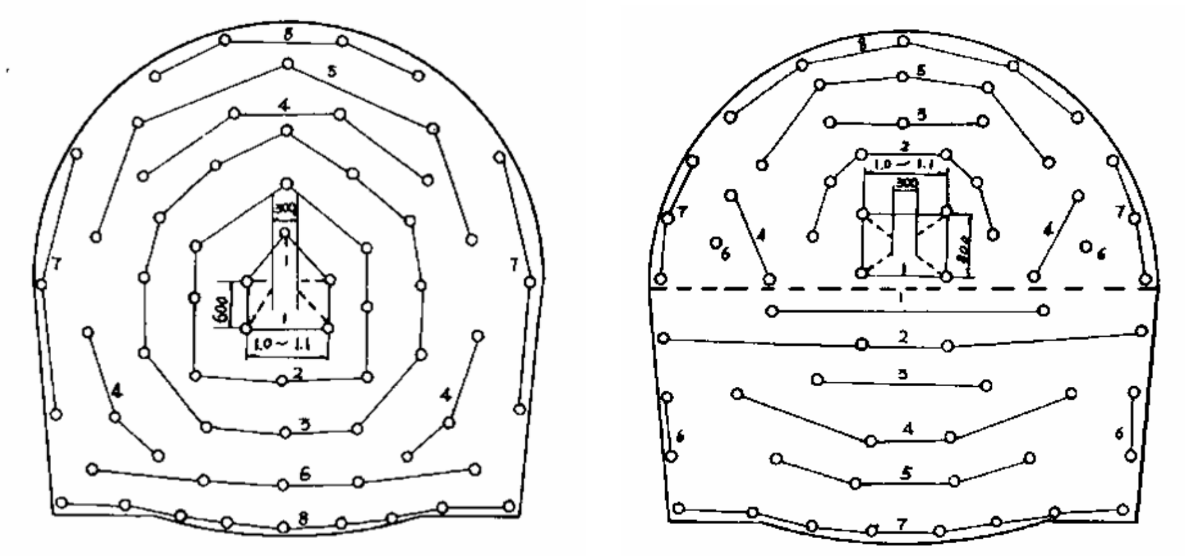
Table 2.1: Excavation Methods of Tunnel (Source: Japanese Standard for Mountain Tunneling, 1996)

Excavation Method		Division of Section of Heading	Applicable Ground Conditions	Advantages	Disadvantages
Full Face Method			･Common excavation method for small section tunnel. ･Very stable ground for large section tunnel ($A>50m^2$) ･Fairly stable ground for medium section tunnel ($A \fallingdotseq 30m^2$) ･Unfit for good grounds interspersed with poor ground that may require the change of the excavation method	･Labor saving by mechanized construction ･Construction Management including safety control is easy because of the single- face excavation.	･Full tunnel length cannot necessarily be excavated by full face alone. Auxiliary bench cut will be adopted as required. ･Fragment rocks from the top of the tunnel may fall down with increased energy & additional safety measures are required.
Bench Cut Method	Long Bench Cut	 Bench length> 50m	･Ground is fairly stable, but Full-face excavation is difficult.	･Alternate excavation of top heading and lower bench reduces equipment and manpower needs.	･Alternate excavation system elongates the construction period.
	Short Bench Cut	 D<Bench length≦ 50m	･Applicable to various grounds such as soily ground, swelling ground, and medium to hard rock ground. (The most fundamental and popular method.)	･Adaptable to changes in the ground condition.	･Parallel excavation makes difficult the balancing of cycle time for top heading and bench.

(a) Blasting Control

“Blasting for underground construction purposes is a cutting tool, not a bombing operation”. The quality of blasting can have a major influence upon the amount of damage and it inflicted upon the rock surrounding of a tunnel. A good tunnel blast is one which results in good fragmentation of the rock within the tunnel, a loose and easily able to dig muck pile of limited lateral extent and minimal damage to the rock surfaces around the tunnel. All of these results can be achieved in a single blast if sufficient care is taken with the design of the blast holes

pattern, the charge distribution and the detonation sequence. A necessary part of all blasting operations is the estimation of potential damage. The following Figure 2.16 shows the blasting pattern of full face excavation and bench excavation (KEPCO, 2005).



(a) For full face excavation **(b) For bench excavation**

Figure 2.16: Blasting Pattern (Source: KEPCO, 2005)

(b) Tunnel Supporting System

Initial ground support is usually installed concurrently with the excavation. For drill and blast excavations, initial ground support is usually installed after the round is shot and mucked out and before drilling, loading, and blasting of the next round. Initial ground support may consist of steel ribs, lattice girders, shotcrete, rock dowels, steel mesh, and mine straps. As the quality of the rock increases, the initial support will also fulfill the role of final support. On the other hand, additional support, such as a cast-in-place concrete lining may be installed. The initial and the final ground support then comprise a composite support system. The following Table 2.2 and Table 2.3 show the tunnel supporting system (KEPCO, 2005) and suggested support condition for various rocks (Albert D. Parker, 1970).

Table 2.2: Tunnel Supporting System

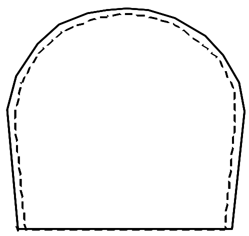
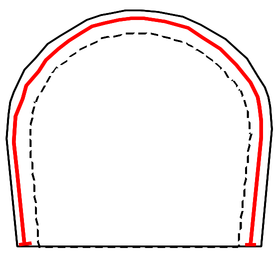
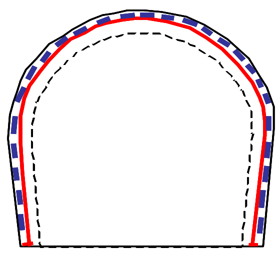
For good rock	For cracky rock	For poor rock
		
- Full excavation - No support - Wire mesh against falling small rocks, if necessary	- Full excavation - Installation of steel ribs - Installation of wooden lagging, if necessary	- Installation of steel ribs and wooden Lagging ASAP

Table 2.3: Suggested Support Condition for Various Rocks

Rock Conditions	Suggested support type
Sound rock with smooth walls created by good blasting. Low in situ stresses.	No support or alternatively, where required for safety, mesh held in place by grouted dowels or mechanically anchored rockbolts, installed to prevent small pieces from falling.
Sound rock with few intersecting joints or bedding planes resulting in loose wedges or blocks. Low in situ stresses.	Scale well then install tensioned, mechanically anchored bolts to tie blocks into surrounding rock. Use straps across bedding planes or joints to prevent small pieces falling out between bolts. In permanent openings, such as shaft stations or crusher chambers, rockbolt should be grouted with cement to prevent corrosion.
Sound rock damaged by blasting with a few intersecting planes. Low in situ stresses	Chain link or weld mesh held by tensioned mechanically anchored rockbolts, to prevent falls of loose rock. Attention must be paid to scaling and to improving blasting to reduce amount of loose rock.

Rock Conditions	Suggested support type
Closely jointed blocky rock with small blocks ravelling from surface causing deterioration if unsupported. Low stress conditions.	Shotcrete layer, approximately 50 mm thick. Addition of micro-silica and steel fibre reduces rebound and increases strength of shotcrete in bending. Larger wedges are bolted so that shotcrete is not overloaded. Limit scaling to control ravelling. If shotcrete not available, use chain-link or weld-mesh and pattern reinforcement such as split sets or Swellex.
Stress-induced failure in jointed rock. First indication of failure due to high stresses are seen in borehole walls and in pillar corners.	Pattern support with grouted dowels or Swellex. Split sets are suitable for supporting small amounts of failure. Grouted tensioned or untensioned cables can be used but mechanically anchored rockbolts are less suitable for this application. Typical length of reinforcement should be about ½ the span of openings less than 6 m and between ½ and 1/3 for spans of 6 to 12 m. Spacing should be approximately ½ the dowel length. Support should be installed before significant movement occurs. Shotcrete can add significant strength to rock and should be used in long-term openings (ramps etc.). Mesh and straps may be required in short-term openings (drill-drives etc.)
Drawpoints developed in good rock but subjected to high stress and wear during blasting and drawing of stopes.	Use grouted rebar for wear resistance and for support of drawpoint brows. Install this reinforcement during development of the trough drive and drawpoint, before rock movement takes place as a result of drawing of stopes. Do not use shotcrete or mesh in drawpoints – place dowels at close spacing in blocky rock.

Rock Conditions	Suggested support type
Fractured rock around openings in stressed rock with a potential for rockbursts	Pattern support required but in this case some ‘flexibility’ required to absorb shock from rockbursts. Split sets are good since they will slip under shock loading but will retain some load and keep mesh in place. Grouted dowels and Swellex will also slip under high load but some face plates may fail. Mechanically anchored bolts are poor in these conditions. Lacing between heads of reinforcement helps to retain rock near surface under heavy rock bursting.
Very poor rock associated with faults or shear zones. Rock-bolts or dowels cannot be anchored in this material.	Fibre-reinforced shotcrete can be used for permanent support under low stress conditions or for temporary support to allow steel sets to be placed. Note that shotcrete layer must be drained to prevent buildup of pressure behind the shotcrete. Steel sets are required for long-term support where it is evident that stresses are high or that roc is continuing to move. Capacity of steel sets estimated from amount of loose rock to be supported.

Installing steel and wooden supports in a tunnel is one of the oldest methods. Many years ago, wooden supports were used exclusively for tunnel support. In later years, steel ribs took the place of wood and, most recently, steel lattice girders are being used in conjunction with shotcrete. If the anticipated rock loads are too great, such as in faulted or weather ground, steel supports may be required. The ribs are assembled from the bottom to certain that the rib has adequate footing and lateral rigidity. Lateral spacer rods (collar braces) are usually placed between ribs to assist in the installation and provide continuity between ribs.

If the joint systems in a hard rock mass intersect in such a way that blocks or wedges are released to fall or slide from the roof or walls of a tunnel, a restraining force must be applied in order to prevent failure. In this case, the simplest solution is to install a series of anchored

rockbolts. Grouting the bolts after tensioning ensures that the loads will be maintained in the bolts. For very severe squeezing conditions, grouted fiberglass dowels are added for face reinforcement and forepoles or similar reinforcing elements are used to pre-reinforce the rock mass ahead of the advancing face. The following Figure 2.17 shows the support system of tunneling work (USACE, 1997).

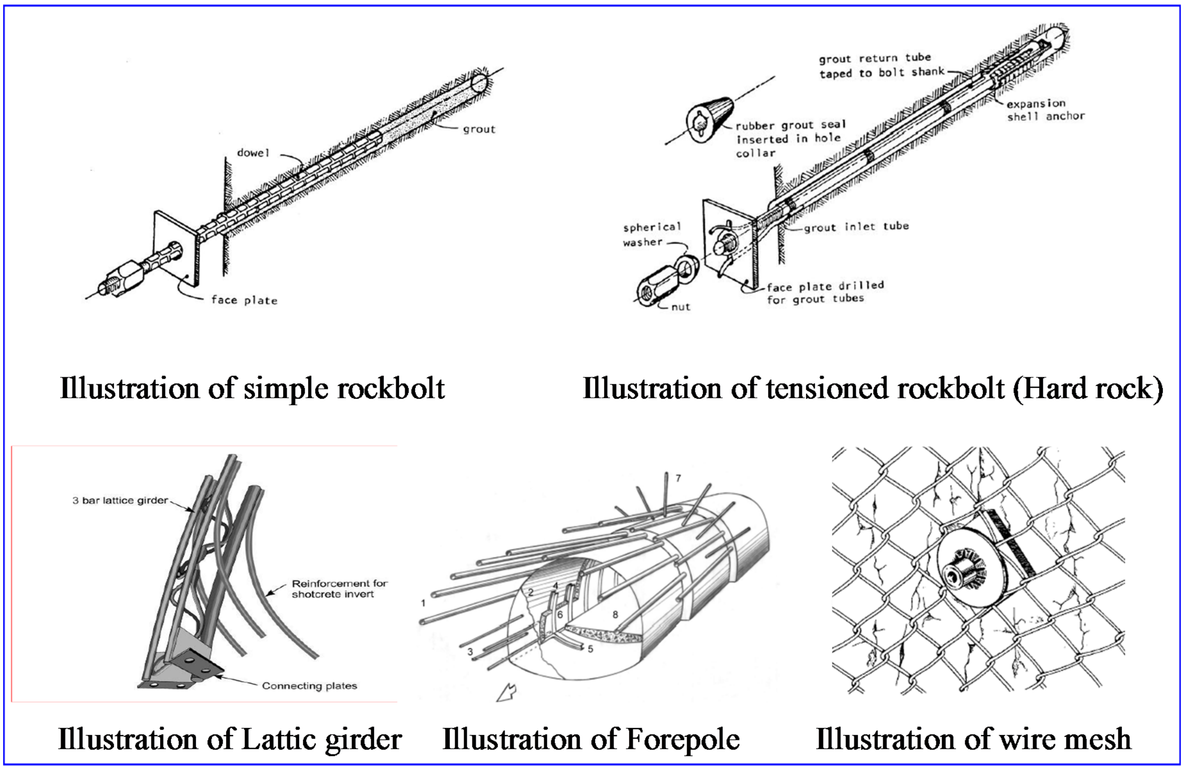


Illustration of simple rockbolt | Illustration of tensioned rockbolt (Hard rock)

Illustration of Lattic girder | Illustration of Forepole | Illustration of wire mesh

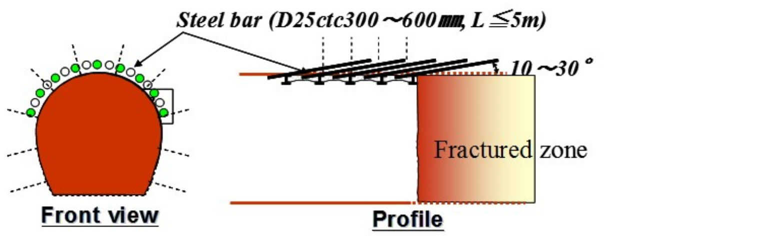
Figure 2.17: Support System Used in Tunneling Work (Source: USACE, 1997)

(c) Auxiliary Methods of Tunneling

Under the situation of severe geological conditions, auxiliary methods are necessary to accompany with major supporting system to overcome the difficulties of tunnel excavation. Auxiliary methods are illustrated in the following Figure 2.18 (KEPCO, 2004):

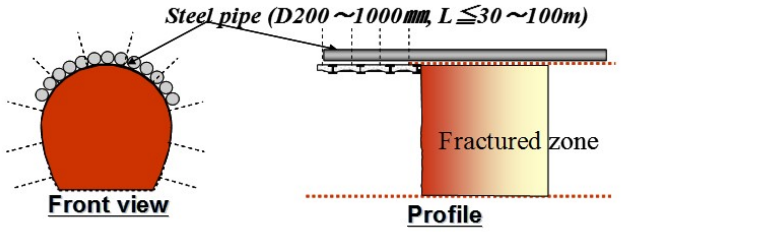
1) *Fore-poling*

For crown stability, rock bolts are set at the crown of tunnel before excavating.



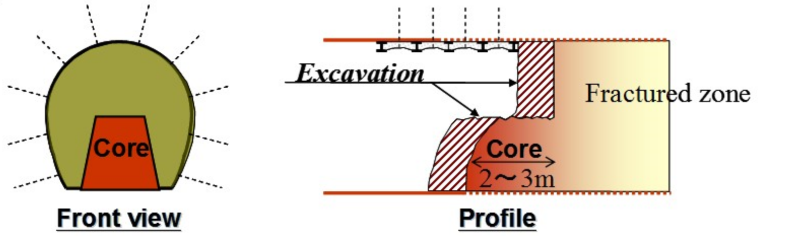
2) *Pipe-roofing*

For crown stability, steel pipes are horizontally placed along the periphery of tunnel section before excavation.



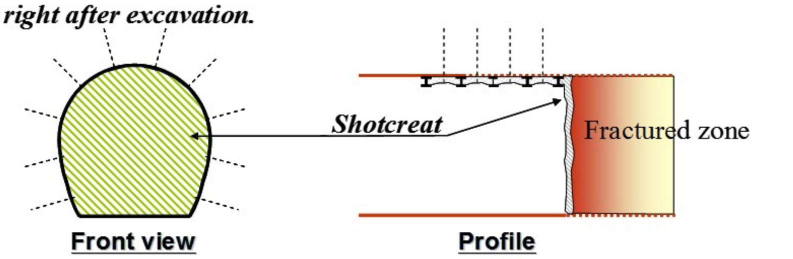
3) *Ring-cutting*

For face stability, a core is remained un-excavated to form a ring-shaped face.



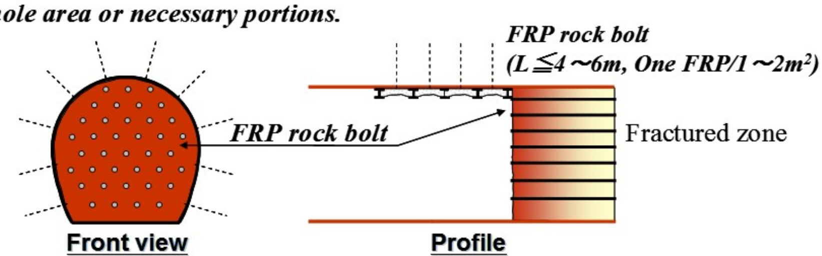
4) *Face-shortcreting*

For face stability, shotcrete 3 ～ 10 ㎝ in thickness is implemented on the face right after excavation.

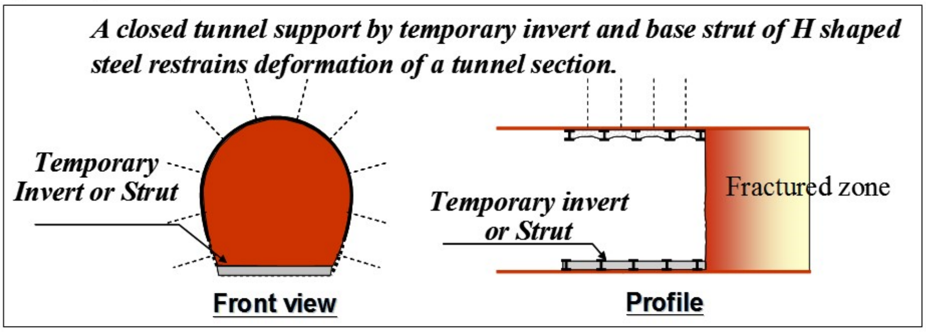


5) *Face-bolting*

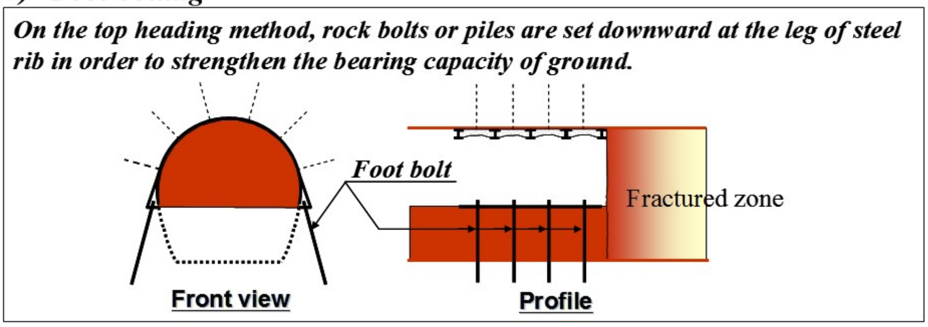
For face stability, FRP(Fiber reinforced plastics) rock bolts are set at the whole area or necessary portions.



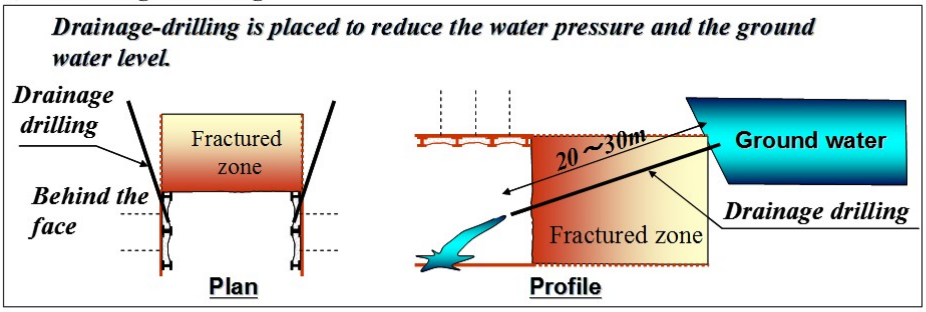
6) Temporary invert



7) Foot-bolting



8) Drainage-drilling



9) Grouting

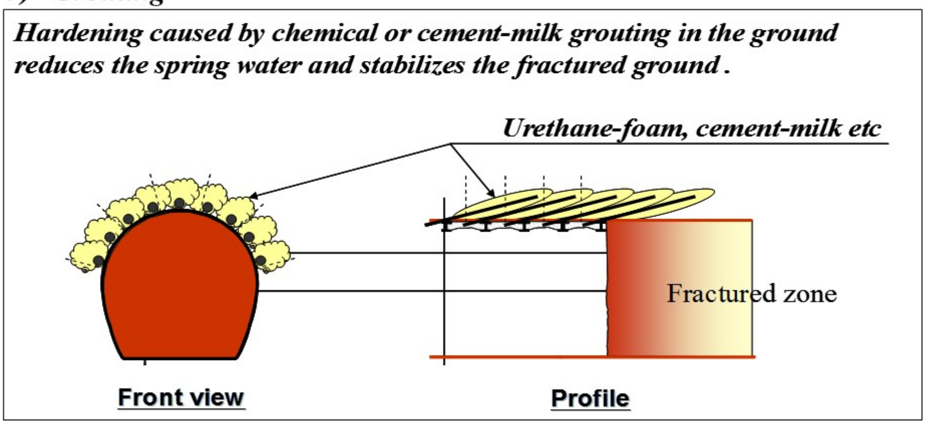


Figure 2.18: Auxiliary Methods on Tunneling Procedures (Source: KEPCO, 2004)

Among those auxiliary methods, some were frequently used such as fore piling, ring cut, in tunneling of Sittaung valley hydropower projects where fractured zone are encountered or excess spring water comes out from tunnel face as tabulated in Table 2.4.

Table 2.4: Auxiliary Methods for Tunnel Excavation

Items	For Stability of			Spring water	Rock Classification		
	Crown	Side wall &Invert	Face		Hard	Soft	Fractured zone
Fore-poling	◎		O		O	◎	◎
Pipe-roofing	Δ		Δ			Δ	Δ
Ring-cutting			◎			◎	◎
Face shotcreting			◎		O	◎	◎
Face-bolting			O		O	O	O
Foot-bolting		O				O	O
Temporary invert		O				O	O
Drainage-drilling	O		O	◎	◎	◎	◎
Grouting	O	O	O	◎	O	O	O

◎: Frequently adopted, *O: Sometimes adopted,* *Δ: Special equipment required.*

2.3.4 Multi Parameter Rock Mass Classification Schemes

The geomechanics classification has found wide applications in various types of engineering projects, such as tunnels, slopes, foundations, and mines. Most of the applications have been in the field of tunneling (Bieniawski 1984). The following are detailed explanations of procedure for measurement and calculation of RQD, RMR and Q System.

The rock quality designation (RQD) index was introduced over 20 years ago as an index of rock quality at a time when rock quality information was usually available only from the geologists' descriptions and the percentage of core recovery (Deere and Deere, 1988). It is aim to provide a quantitative estimate of rock mass quality, and equal to the percentage of intact core pieces longer than 100mm in the total length of core as illustrated in Figure 2.19. Palmstron (1975) proposes the following Eq. (2.1) for measuring the RQD in tunnel.

$$RQD = 115 - 3.3 \times Jv \qquad (2.1)$$

Jv: numbers of joints in $1m^3$ and to get Jv:

1) The numbers of joint system are measured. (For instance, there are 3 joint systems.)

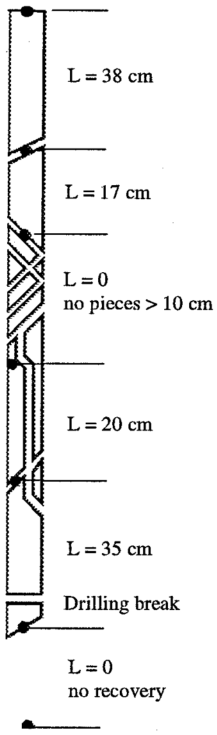
2) The spacing among the joint systems is measured in meter. (For instance, they are 0.5, 0.2m and 0.1m.)

3) Jv is calculated as follows:

$$Jv = 1/0.5 + 1/0.2 + 1/0.1 = 17/m^3$$

4) RQD = 115 – 3.3 x 17 = 60 (approximately)

5) In case of Jv< 4.5, RQD is to be 100.



Total length of core run = 200 cms

$$RQD = \frac{\sum \text{Length of core pieces} > 10\,\text{cm length}}{\text{Total length of core run}} \text{ X } 100\%$$

$$RQD = \frac{38+17+20+35}{200} \text{ X } 100\% = 55\%$$

RQD (%)	Rock Quality
<25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent

Figure 2.19: Procedure for RQD (Source: Neil Benson)

Bieniawski (1989) defined that due to the RMR system having been modified several times, and since the method is interchangeably known as the geomechanics classification or the Rock Mass Rating system, it is important to state that the system has remained essentially the same in principle despite the changes. The following six parameters are used to classify a rock mass using the RMR system (Geomechanics Classification):

1) Uniaxial compressive strength of rock material.

2) Rock quality designation (RQD).

3) Spacing of discontinuities.

4) Condition of discontinuities.

5) Groundwater conditions.

6) Orientation of discontinuities.

In the procedure of Rock Mass Rating System, the following step have to apply as illustrated in Figure 2.20:

- Rock mass divided into structural regions
- Each region is classified separately
- Boundaries can be rock type or structural, eg: faults, dykes, shear zones, etc.
- Can be sub divided based on significant changes, eg: discontinuity spacing.

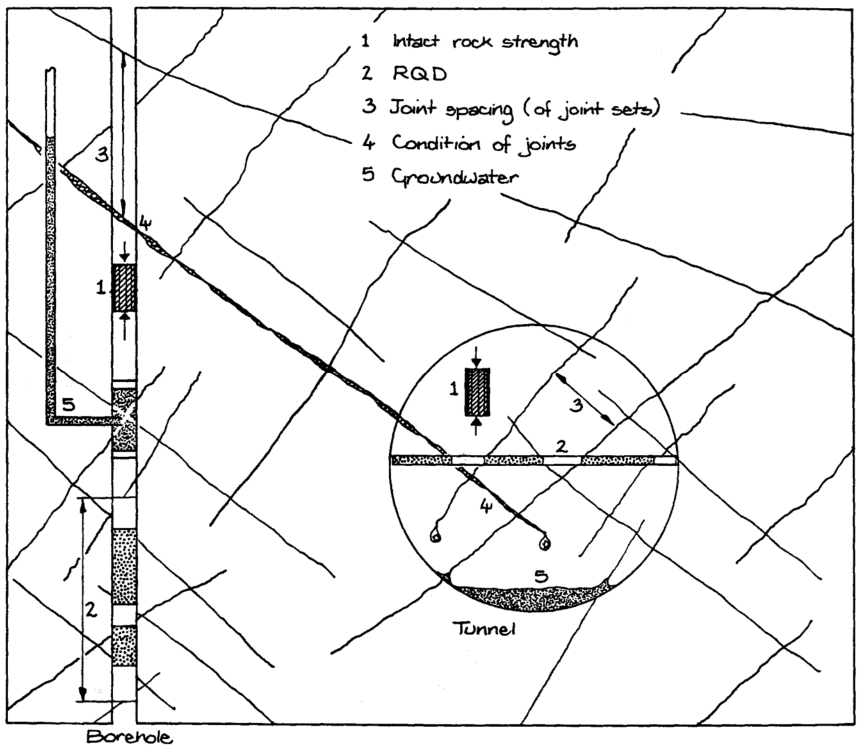


Figure 2.20: Procedure for Rock Mass Rating System (Source: Neil Benson)

But it was improved by Bienawski (1976 to 1989) as follow and development of rock mass rating system is shown in Table 2.5 (Neil Benson (Golder Associates)).

- System refined by greater data
- Ratings for parameters changed
- Adapted by other workers for different situations
- Project specific systems.

Table 2.5: Development of Rock Mass Rating System

Item	Value	1973	1974	1976	1979	1989
Point load index	7 MPa	5	5	12	12	12
RQD	70%	14	14	13	13	13
Spacing of discontinuities	300 mm	20	20	20	10	10
Condition of discontinuities	Described	12	10	20	20	25
Groundwater	Dry	10	10	10	15	15
Joint orientation adjustment	Very favourable	15	15	0	0	0
	RMR	76	74	75	70	75

After Bieniawski (1979), guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system is tabulated in Table 2.6, and geomechanics classification is presented in Table 2.7 (Neil Benson (Golder Associates)).

Table 2.6: Guidelines for Excavation and Support of 10 m Span Rock Tunnels

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Table 2.7: The Rock Mass Rating System (Geomechanics Classification of Rock Masses)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core *Quality RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

Bieniawski (1989) mentioned the Q system of rock mass classification was developed in Norway in 1974 by Barton, Lien, and Lunde, all of the Norwegian Geotechnical Institute. Its development represented a major contribution to the subject of rock mass classification for a number of reasons: the system was proposed on the basis of an analysis of 212 tunnel case histories from Scandinavia, it is a quantitative classification system, and it is an engineering system facilitating the design of tunnel supports.

The Q system is based on a numerical assessment of the rock mass quality using six different parameters:

1) RDD.
2) Number of joint sets.
3) Roughness of the most unfavorable joint or discontinuity.
4) Degree of alteration or filling along the weakest joint.
5) Water inflow.
6) Stress condition.

These six parameters are grouped into three quotients to give the overall rock mass quality Q is as following Eq. (2.2) (After Barton et al. 1974):

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{2.2}$$

where,	RQD	is the Rock Quality Designation
	J_n	is the joint set number
	J_r	is the joint roughness number
	J_a	is the joint alteration number
	J_w	is the joint water reduction factor
	SRF	is the stress reduction factore

The rock quality can range from Q = 0.001 to Q = 1000 on a logarithmic rock mass quality scale.

Bieniawski (1989) explained that from the above equation, the first two parameters represent the structure of the rock mass and crude measure of block or particle size. The quotient of

the third and the fourth parameters is said to be roughness and frictional characteristics of joint walls or infill material. The fifth parameter is a measure of water pressure, while the sixth parameter is a measure of a) loosening load as excavated through shear zones b) rock stress in competent rock, and c) squeezing loads in plastic incompetent rock. This sixth parameter is regarded as the "total stress" parameter. The quotient of the fifth and sixth parameters describes the "active stress".

Barton et al. (1974) considered the parameter Jn, Jr, and Ja as playing a more important role than joint orientation, and if joint orientation had been included, the classification would have been less general. However, orientation is implicit in parameters Jr, and Ja because they apply to the most unfavorable joints.

Classification of individual parameters used in the Tunneling Quality Index Q is presented in Table 2.9 (Neil Benson (Golder Associates)).

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter which they called the Equivalent Dimension, De, of the excavation. By a quantity called the Excavation Support Ratio, ESR, Hence:

$$D_e = \frac{Excavation\ span{,}diameter\ or\ height\ (m)}{Excavation\ Support\ Ratio\ ESR} \qquad (2.3)$$

The value of ESR is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al. (1974) suggested the following values as tabulated in Table 2.8:

Table 2.8: The Suggested Values of Excavation Support Ratio

Excavation category		*ESR*
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Table 2.9: Classification of Individual Parameters Used in the Tunneling Quality Index Q

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	***RQD***	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	minimum strength.
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
a. Rock wall contact		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16

Table 2.9: Classification of Individual Parameters Used in the Tunneling Quality Index Q (Continued)

DESCRIPTION	VALUE	NOTES	
4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		***SRF***	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

Table 2.9: Classification of Individual Parameters Used in the Tunneling Quality Index Q (Continued)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\,J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Estimated support categories based on the tunneling quality index Q is illustrated in Figure 2.21. In the case of rock foundations, knowledge of the modulus of deformability of rock masses is of prime importance. The geomechanics classification proved a useful method for estimating in-situ deformability of rock masses (Bieniawski, 1978). RMR and Q system or variants are the most widely used on both incorporate geological, geometric and design/engineering parameters to obtain a “value” of rock mass quality by using empirical and require subjective assessment. As shown in Figure 2.22, the correlation of prediction of in-situ deformation modulus E_m was obtained.

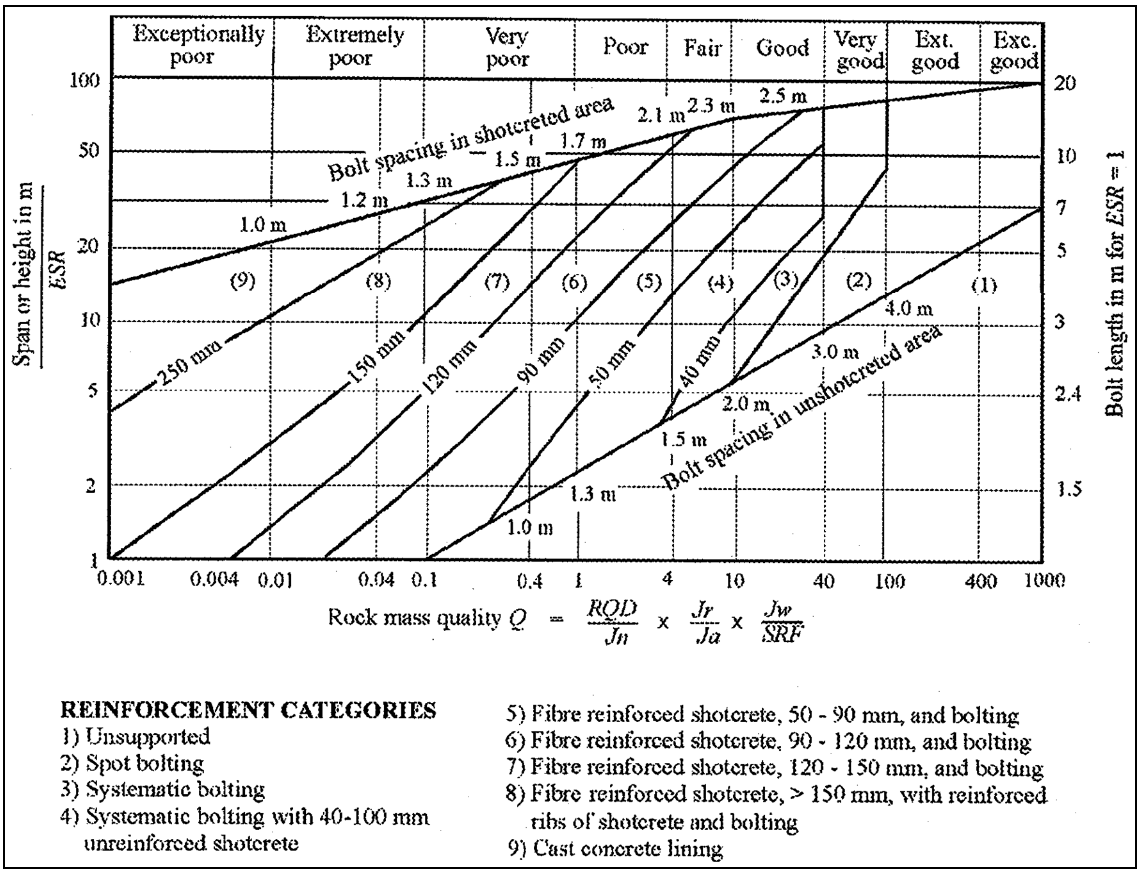


Figure 2.21: Estimated support categories based on the tunneling quality index Q (Source: Neil Benson)

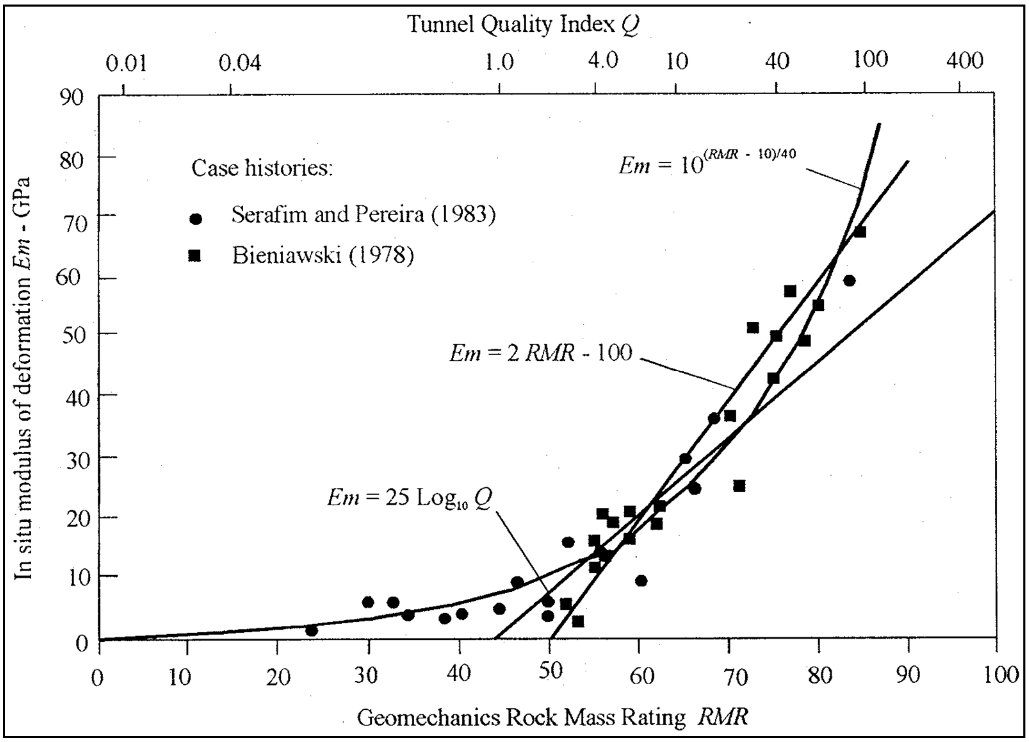


Figure 2.22: Prediction of In-situ Deformation Modulus E_m from Rock Mass Classifications (Source: Neil Benson)

As illustrated in Figure 2.23, a correlation was developed between the Q index and the RMR (Bieniawski, 1976) which has a total of 111 case histories were analyzed for this purpose and it can be seen that the following relationship Eq. (2.4) is applicable:

$$RMR = 9 \ln Q + 44 \tag{2.4}$$

The above correlation was further substantiated by Jethwa et al. (1982), whose case studies are also included in Figure 2.22. Further comparisons between the Q and the RMR systems are given by Barton (1988).

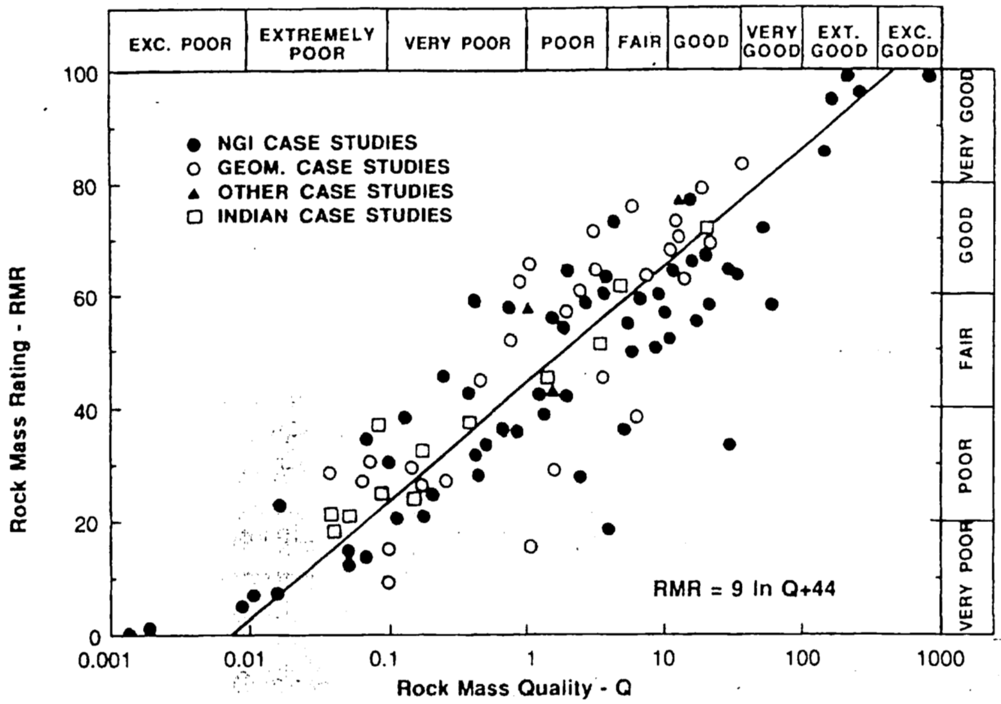


Figure 2.23: Correlation between the RMR and the Q Index

(Source: Bieniawski, 1989)

2.3.5 Geological Accept on Tunneling

Rock and soil are not man-made and their properties can vary greatly over short distances. In reality, many geological characteristics cannot be quantified precisely and intelligent guesses based upon experience and logical arguments are the best that can be hoped. For a tunnel, most important during construction is the instantaneous water inflow at any given location and the reduction of inflow with time. Realistic prediction of the geological conditions and continuous surveying during construction is essential for tunneling. In general, the behavior and foreseeable problem of rock mass in which tunnel excavation took place is mentioned as following Table 2.10 (Albert D. Parker, 1970).

Table 2.10: Rock Types, Conditions and Stability Problems of Tunneling

Rock type and conditions	Response to tunneling and stability problems
Massive or bedded lime stones. Marbles	Simple tunneling conditions. Structurally controlled failures, mainly controlled by rock bolts.
Filled karstic voids	Risk of collapses. Probing ahead essential and use of spiles or a fore pole umbrella to cross void.
Sandstone flysch	Gravity driven structurally dependent instability in low stress environments and occasionally stress dependent instability when strength to stress ratio is low.
Siltstone flysch and shales	Stress dependent instability resulting in significant deformations and minor face instability. Control of deformation is essential and both temporary and permanent inverts may be required to form a load bearing shell.
Sheared and chaotic flysch	Squeezing conditions and face instability problems at depth. Control of deformations is essential and, to control extreme squeezing, yielding support may be required.
Sound ophiolites (peridotites and gabbros)	Structurally dependent instability, more severe when discontinuities are serpentinised. Block size normally irregular and this requires a conservative excavation and support approach.
Sheared serpentinites and ophioliticmelanges	Squeezing conditions at depth (e.g. more than 200m). Control of deformations is essential and, to control extreme squeezing, yielding support may be required.
Molasses (tectonically undisturbed sedimentary sequence of rocks)	Simple tunneling conditions. Gravity driven instability under low stress. Under confined conditions brittle failure can occur in high stress environments. Weak geotechnical conditions in the weathered surface layers, slope stability issues in portals.
Gneiss schists	Simple tunneling conditions if not heavily tectonized and/or weathered. Structurally dependent instability.
Phyllites	Weak rock tunneling. Deformation problems in cases of deep tunnels. Control of deformation is essential, both temporary and permanent inverts are generally required to form a load bearing shell.

Rock type and conditions	Response to tunneling and stability problems
Tectonics breccia in brittle rocks, kataclastes	Raveling due to loss of interlocking as confinement is released at face. Maintaining confinement is important and this can generally be achieved by retaining a core at the face and the use of pre-reinforcement elements

Main causes of tunnel collapse in geological concern are:

- Weak or fractured geological condition
- Change of geological condition.
- Increase of spring water amount.

To avoid these problems detail investigation, control and solution are recommendable prior to excavation of tunnel. A good engineering geologist and a good geotechnical engineer, working as a team, can usually make realistic educated guesses for each of the parameters required for a particular engineering analysis. Geological condition to be considered in Tunnel excavation is summarized as following Table 2.11 (Albert D. Parker, 1970).

Table 2.11: Geological Problems of Tunneling

Geological condition	Causes of collapse
Swelling rock	- Swelling of excavated tunnel face - Acting of large earth load - Long-term deformation and increase of earth load - Deformation of tunnel support or concrete lining
Unconsolidated rock	- Outflow of ground by spring water - Decline of supporting function by rock weakening - Large amount of spring water and increase of water pressure
Thin overburden (Overburden < 3D)	- Settlement of ground surface -Acting of large earth load -Deformation of tunnel support or concrete lining
Landslide or loosen Ground around Tunnel portal	- Acting of unsymmetrical load - Deformation of tunnel support or concrete lining - Settlement of H-beam support - Landslide or collapse

2.4 Geology Assessment on Tunneling

2.4.1 Introduction

A basic principle in tunneling is that "a tunnel should be supported by the surrounding ground as much as possible". That is, a tunnel must be sustained by the shear strength of the ground. Steel supports, shotcrete lining, rock bolts and secondary lining should only play a role of assistant members in maintaining the strength of the ground (Adachi, 2001) as illustrated in Figure 2.24.

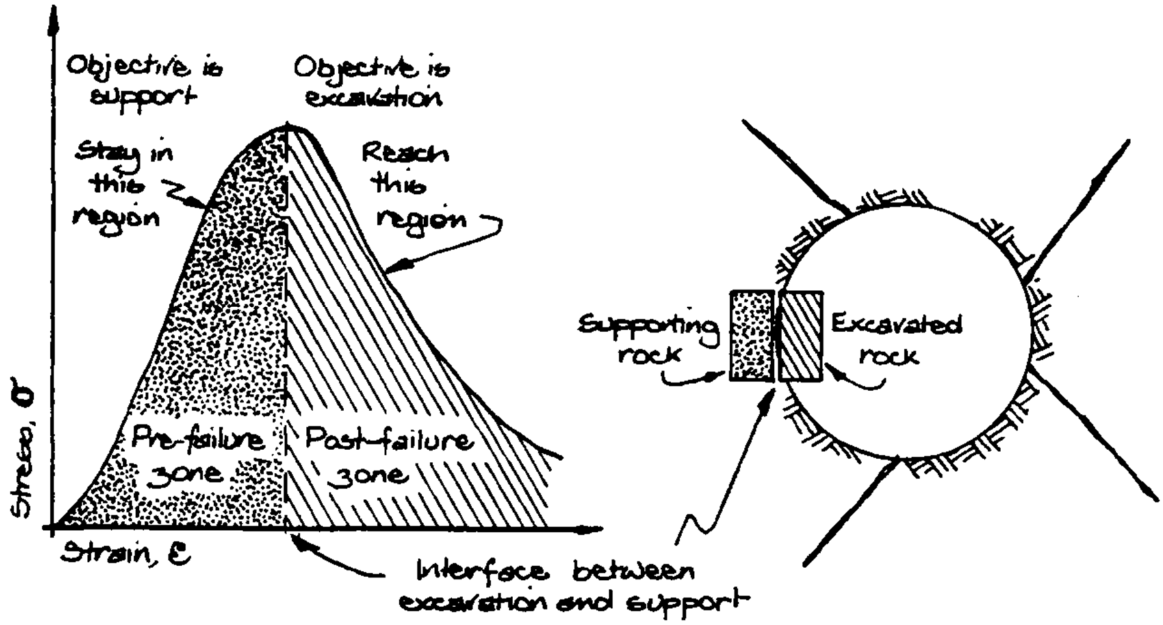


Figure 2.24: Behavior of Rock in Tunnel Excavation (Source: Neil Benson)

2.4.2 Engineering Rock Mass Classification for Tunnel

Classifications have played an indispensable role in engineering for centuries. In rock engineering, the first major classification system was proposed over 40 years ago for tunneling with steel supports (Terzaghi, 1946). Considering the three main design approaches for excavation in rock– analytical, observational, and empirical– as practiced in mining and civil engineering, rock mass classifications today from an integral part of the most predominant design approach, the empirical design methods. Indeed, on many underground construction and mining projects, rock mass classifications have provided the only systematic design aid in an otherwise haphazard "trial-and error" procedure. Rock mass classifications were developed to create some order out of the chaos in site investigation procedures and to provide the desperately needed design aids. They were not intended to

replace analytical studies, field observations, and measurements, nor engineering judgment (Bieniawski, 1989).

Bieniawski (1989) suggested that in essence, rock mass classifications are not to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology. When used correctly and for the purpose for which they were intended, rock mass classifications can be powerful aids in design. The objectives of rock mass classifications are therefore to

1) Identify the most significant parameters influencing the behavior of rock mass.
2) Divide a particular rock mass formation into groups of similar behavior, that is, rock mass classes of varying quality.
3) Provide a basis for understanding the characteristics of each rock mass class.
4) Relate the experience of rock conditions at one site to the conditions and experience encountered at others.
5) Derive quantitative data and guidelines for engineering design.
6) Provide a common basis for communication between engineers and geologists.

The preceding items suggest the three main benefits of rock mass classifications:

1) Improving the quality of site investigations by calling for the minimum input data as classification parameters.
2) Providing quantitative information for design purposes.
3) Enabling better engineering judgment and more effective communication on a project.

2.5 Evaluation on Geological Data of Tunneling

Bieniawski, (1989) mentioned that considering the three main design approaches for excavation in rock – analytical, observational, and empirical– as practiced in mining and

civil engineering. Rock mass classifications were developed to create some order out of the chaos in site investigation procedures and to provide the desperately needed design aids.

Among the design methods which are available for assessing the stability of tunnels, empirical design methods which is widely used in the tunneling, assess the stability of tunnels by the use of statistical analyses of underground observations. Engineering rock mass classifications constitute the best-known empirical approach for assessing the stability of underground openings in rock (Goodman, 1980; Hoek and Brown, 1980).

In the case study process of tunnel excavation, one tunnel demonstrates the role of rock mass classifications in tunnel design specifications, while the other makes comparisons of classifications with monitoring data, and then evaluates the comparison of actual results with expected results by using.

2.6 Summary

From the literature review, it is noticed that scale of hydropower project and concerning about geo-risk on underground structure, tunnel, has many uncertainty and difficult to handle from feasibility to implementation process, and it is essential for geo risk reduction by proper construction management, advancing geological investigation and evaluation of rock quality which can improve tunnel excavation and supporting system on weak geological area. The followings are summary of lecture review:

- The uncertainty associated with geologic conditions on tunnel structure and the associated impacts on hydropower construction productivity is significant. So application of risk assessment and risk response on underground construction works for hydropower projects is essential.
- Proper design methodology has to apply on tunneling and empirical approach is commonly used in most of tunnel structures all over the world. It is preferable for geo-risk reduction but technical knowhow on geomechanics and better construction management is essentially required.
- According to statistical analyses of underground observations, design methodology as related to rock mass classification is important and improvement of geological investigation technics are also partially required for better estimation on mountain tunneling.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

The third chapter describes the guideline which tells a detailed plan or explanation in overall methodology of geo-risk reduction on mountain tunneling of hydropower project. It comprises geo-risk identification and risk control on tunneling of hydropower projects by means of empirical method. It includes three main parts and step by step procedures can be sketched in systematic way as follows:

1) Comparative study on tunneling practice of hydropower project for detailed of risk assessment.

2) Identification, classification and assessment of geo-risk on cost and schedule of the projects.

3) Risk control by human factors and mechanical factors as responses on tunneling of hydropower projects.

The methodology of thesis is mainly focus on geo-risk of hydropower tunneling and impact on project cost and construction schedule. Furthermore, it proposes the human factors and mechanical factors which can improve the hydropower engineering capacity building and tunneling methodology for excavation and supporting system on weak geological area. The risk management framework is already described in Chapter 2.

3.2 Goal and Scope of Geo-Risk Management on Tunneling

The main goal of this study is to figure out unforeseen geological risks of tunneling and how to impact the cost and schedule on construction of hydropower projects. Unforeseen geological risk plays an important role in the hydropower development; sometimes, it may greatly impact to the project not achieving its objectives. Undoubting, geo-risk management will not only secure the financial and schedule burden of the project but also bring good practices of preserving and enhancing project quality and social well-beings in hydropower development.

Flanagan and Norman (1993) mentioned that it is important to distinguish the sources of risk from their effects. Ultimately, all risk encountered on a project is related to one or more of the following:

- Failure to keep within the cost budget/ forecast/ estimate/ tender;
- Failure to keep within the time stipulated for the approvals, design, construction and occupancy;
- Failure to meet the required technical standards for quality, function, fitness for purpose, safety and environment preservation.

The overall framework and scope of research study is based on the geo-risk evaluation of tunnel excavation on hydropower projects, which is caused by human factors and mechanical factors in Myanmar. The research methodology of scope and frame work of the study is illustrated in following figures:

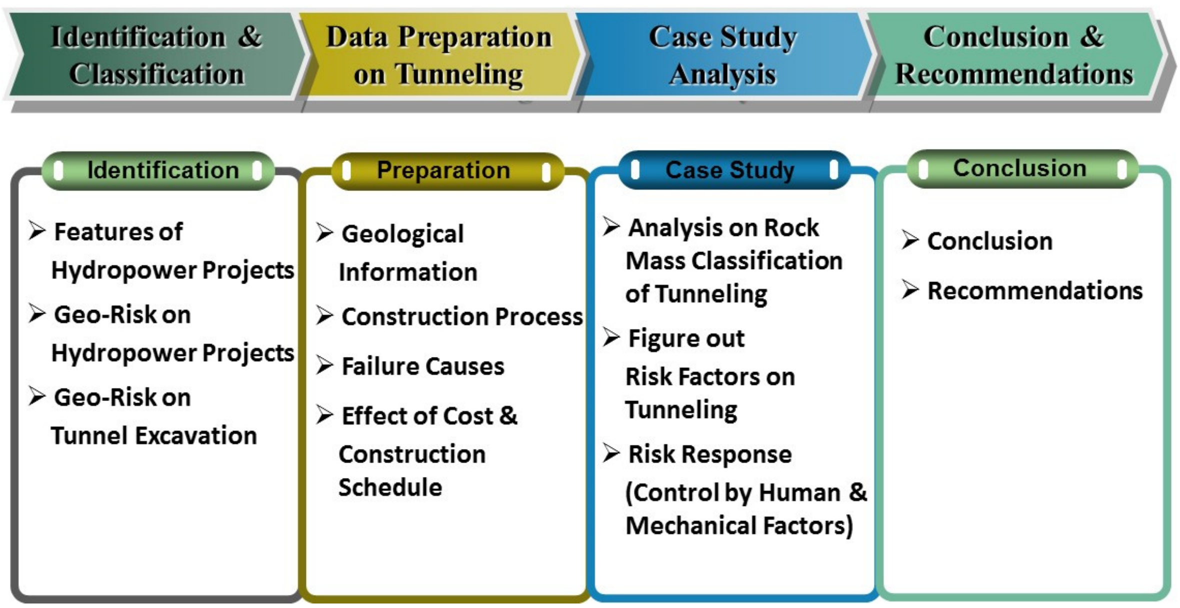


Figure 3.1: Scope and Frame Work of the Study

3.3 Selection of Study Area

Nowadays, the construction of hydropower projects is one of the priorities for development of nation as current government strategies. Therefore, Ministry of Electric Power (MOEP) had been trying to implement large scale hydropower projects to fulfill the electricity requirement of the country. Most of projects are included tunnel works those for power tunnel, diversion tunnel and access tunnel etc. The following Figure 3.2 shows the status of tunneling in hydropower projects of Myanmar. In the region of hard rock, tunneling of the

projects are simple, but the tunnel construction in poor geology face much complicated disturbances leading to collapse especially for Sittaung valley hydropower projects in Myanmar.

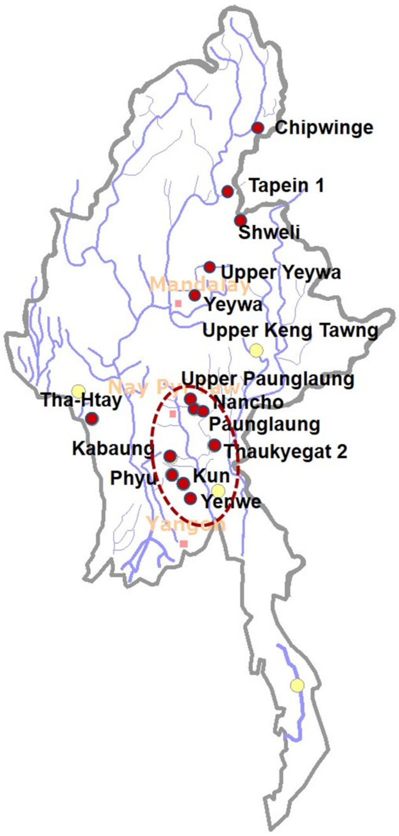


Figure 3.2: Tunneling Practice on Hydropower Projects in Myanmar (Source: KEPCO, 2008)

Therefore, this research focuses on some projects comparative study which are located in the complex geological area. Among the Sittaung Valley Projects, the upper most three projects are located in very good geological area, and middle and lower other seven projects are located in complex geological area which engineering challenges of different tunneling methods in different geological conditions of rocks and solving the problems daily encountered during under construction. Among the ten projects of Sittaung valley river basin, four projects are selected for case study which are seriously impact on cost overrunning, and construction delaying in some projects by means of geo-risk failure mechanisms.

3.4 Methodology on Geo-Risk Management

To access risk, there are two types of risk: "Subjective Risk" based on questionnaires, interviews, brainstorming among experts and so on, and "Objective Risk" evaluated

quantitatively by using probabilistic and /or stochastic models (Ohtsu, 2012). In general, geo-risk management comprises four phases; “Risk Identification”, “Risk Classification,”, “Risk Assessment” and “Risk Response”. The risk management framework which shows the sequent for dealing with risk is illustrated in Figure 3.3.

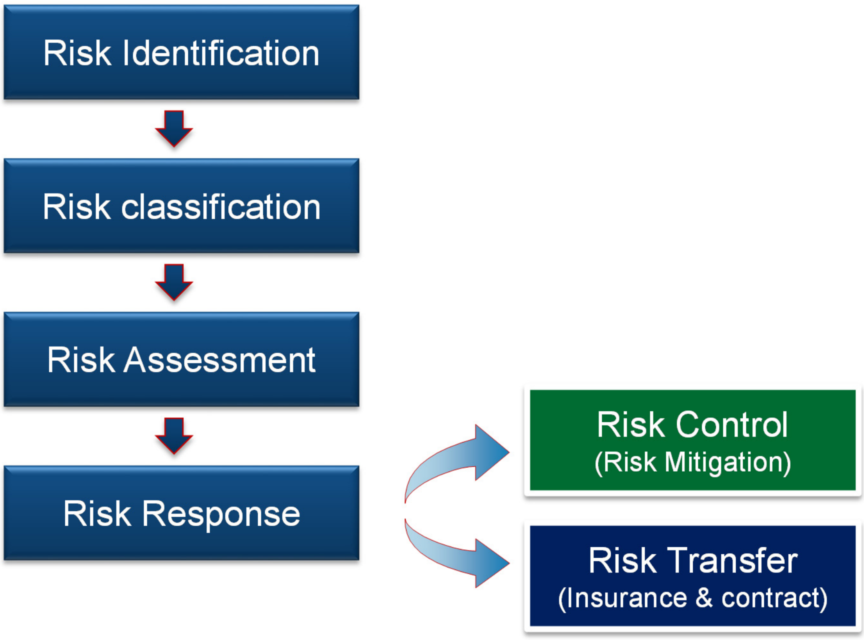


Figure 3.3: Flow Chart of Risk Management (Source: Ohtsu, 2013)

3.4.1 Risk Identification on Tunneling

First, the paper identifies that geo-risk factors involved in tunnel construction are mainly divided into two parts: “geological condition” and “construction management system”, which are perceived as “Natural Hazard” and “Man-made Hazard”, respectively. Carley and Reminga (2004) defined that improper arrangement of the organization’s structure makes various series of problems affecting on the project-organization.

In this step, some projects was selected which can be available recorded data and familiarity of the project during construction period, for the required studies materials by means of knowledge base and statistical analysis. Then the practical risk factors are judged from the tunneling of hydropower projects and the human factor and mechanical factor will be highlight from the verification process.

The verification risk factors is important for the study because the rationale of organizational management and mechanical geo-risk factor to tunneling process is necessary to find out for the cases study in order to gather the concrete results of the analysis. Therefore, risk factors

identification was from tunnel construction process, failure behavior and geological information for verification of geo-risk in tunneling of hydropower development.

3.4.2 Risk Classification on Tunneling

As for "geological condition", the paper suggests that classification of geological condition whether it is poor or good is a key factor to identify geo-risk factor, which impact cost overrun/ delay of construction period.

Geo-risk analysis on tunneling is included to comprehend the rational of human risk factors and mechanical risk factors. The case study evidences are examined geo-risk impact on cost and schedule by comparative study and the analytical thinking is carried out to investigate the risk factors of human and mechanical on tunneling of weak geological area. Firstly, identification of risk factors is developed by combining the knowledge of literature reviews and statistical evaluation on geo-risk. The objective of risk classification is to figure out the dominant weak points on tunneling of poor or good geological condition and to control the risks by improving the human and mechanical factors which is majorly impact on tunneling of hydropower projects.

3.4.3 Risk Assessment on Tunneling

The paper shows case studies for four tunnels associated with development of hydropower projects in Myanmar, focusing on cost overrun and delay of construction period. As for "geological condition", it points out that there is big difference between rock classifications such as Rock Tunneling Quality Index (Q)/ Rock Mass Rating (RMR) evaluated in prior to excavation and those measured during excavation. At the area where the gap is significant, tunnel collapsed. Risk factors pointed out are follows:

- Limitation of ground investigation data
- Poor evaluation of rock mass
- Poor/ Impropriate construction works
- Poor procurement of tunnel support members
- Lack of finance to procure tunnel support members

In the case of tunnels constructed in good geological condition, while construction management system does not matter from a view point of tunnel stability during construction,

it may lead to cost overrun/ delay of construction period. On the other hand, impropriate prediction of "geological condition" causes significant cost overrun and delay of construction period, and "poor construction management system" increases additional losses.

(a) Projects Selection for Case Study

In this study, mainly focus on excavation process of mountain tunnel which involved in geotechnical risk to shed further light on the cost overrunning and construction delaying caused by mismanagement on human factor and mechanical factor. To make clear geotechnical risk on tunneling, some projects were selected which are similar tunnel structure and different geological conditions, and make a comparison on tunneling progress for those projects. Furthermore, for making clear geo-risk behavior, two projects was selected which are implemented by different parties and same conditions of poor geology, and reviewed on actual recorded geomechanics data of excavated tunnel face. It can be highlight on geo-risk of poor geology and responsibility of organizational management role in tunneling of hydropower projects.

Comparative study is to figure out the significant problems and weak point on tunneling practice of hydropower projects as well as to improve the tunneling procedure for better situation. In this step, data collection for the required studies materials such as project features, geological data, tunnel excavation recorded data, and process of tunneling method which are located in different geological area to identify geo-risk on tunneling of hydropower project. Then the practical risk factors are judged on hydropower projects and the geo-risk management process will be highlight.

(b) Geological Investigation and Rock Mass Classification on Tunneling

In Myanmar, geological investigation on hydropower tunneling is used as traditional way. In order to consider geological features along the tunnel, geologic survey is carried out to know the type of bed rock, and the physical and dynamic characteristics of the bed rock. During feasibility stage, geologic survey is conducted on the tunnel alignment such as site reconnaissance, aerial photograph interpretation, physical prospecting, and exploratory drilling. The following are the general consideration on tunnel planning:

- Function of a tunnel
- Ground conditions (Topography, Geology, Ground water, etc.)

- Conditions of location
- Location of tunnel portals
- Tunnel routes (Plane layouts of tunnel routes)
- Longitudinal gradient
- Internal cross section (Shape, Dimension)
- Safety of construction works
- Construction effects to surrounding environment
- Total costs (Construction cost, operation cost and maintenance cost in future)

By the geologist, site reconnaissance survey is carried out to know the ground conditions such as topography, geology, ground water, etc. Even though subterranean conditions can be assumed by surface geologic and physical prospecting, drilling and exploratory adit are essentially required to confirm the lithology of underground structure. The features of drilling, its applicability and application methods are described below (NEF, 1996).

1) Short construction period and minimal cost to examine underground geologic conditions
2) Possible to drill in any direction
3) Recovered core enables a variety of laboratory tests
4) Drill holes enable a variety of tests including permeability test, varied logging tests, seismic prospecting, ground water test, and bore load test.

Generally, drilling is conducted to acquire information regarding subterranean geologic conditions by testing recovered core and in drill holes, and then geological plan and profile of the tunnel is prepared for prediction of excavation method and supporting system in the prior design stage.

Under the construction stage, geological survey is processed periodically and timely in accordance with tunneling progress in order to grapse present condition of tunnel and to reflect findings in subsequent tunnel predictions, and in design and excavation of tunnel. Detail investigation, control and solution are required prior to excavation of tunnel. A good

engineering geologist and geotechnical engineer, working as a team, can usually make realistic educated guesses for each of the parameters required for a particular engineering analysis on tunneling (Kansai, 2008).

To provide a quantitative estimate of rock mass quality from drill logs and tunnel excavated face, Rock Quality Designation index (RQD) is used as a component in the Rock Mass Rating (RMR) and Rock Tunneling Quality index (Q System) as mentioned detail in Chapter 2. The two empirical RMR system and Q classification system have been widely used in selecting the tunnel excavation methods, rock supporting methods and blasting patterns in the tunneling works of hydropower projects in Myanmar.

(c) Design Methodologies for Tunnel

Bieniawski (1989) summarized the topic of design methodology as related to rock mass classification is important for two reasons. Firstly, rock mass classifications are based on case histories and hence tend to perpetuate conservative practice unless they are seen as a design aid, requiring periodic updating. Secondly, they represent only one type of the design methods, an empirical one, which needs to be used in conjunction with other design methods. A good design methodology can ensure that rock mass classifications are used with greatest effect and that they do not hamper but promote design innovation and state-of-the-art technology. Nevertheless, while extensive research is being conducted in rock mechanics today, there still seems to be a major problem in "translating" the research findings into innovative and concise design procedures.

The design methods which are available for assessing the stability of mines and tunnels can be categorized as follows:

1) Analytical methods.
2) Observational methods.
3) Empirical methods.

Analytical design methods utilize the analyses of stresses and deformations around openings. They include such techniques as closed-form solutions, numerical methods (finite elements, finite difference, boundary elements, etc.), analog simulations (electrical and photoelastic), and physical modeling.

Observational design methods rely on actual monitoring of ground movement during excavation to detect measurable instability and on the analysis of ground-support interaction. Although considered separate methods, these observational approaches are the only way to check the results and predictions of the other methods.

Empirical design methods assess the stability of mines and tunnels by the use of statistical analyses of underground observations. Engineering rock mass classifications constitute the best-known empirical approach for assessing the stability of underground openings in rock (Goodman, 1980; Hoek and Brown, 1980).

Among these three methods, empirical design method is dominantly used for tunneling practice on hydropower projects in Myanmar.

3.4.4 Risk Response on Tunneling

Based on above study, it is noticed that the most geo-risk behaviors on weak geology among the implementation of hydropower projects are not only depend on weak geology but also mismanagement on human factors. So it is essential for geo risk reduction by improving human factors and mechanical factors.

In order to scope with difficulties associated "poor construction management system" in Myanmar, following remedial measure would be expected.

- Skill of construction works.
- Decision-making system in response to realization of unexpected poor geological condition.
- Procurement system.
- Financial system.

For the human factors, it is important to establish a capacity building for hydropower engineers and skilled labors by training in-house engineers and workforce which can promote institutions in construction engineering of hydropower projects. The most efficient way of structuring for capacity building is education in technical colleges and universities which plays an important role to increase the skilled engineers. In the future of hydropower development, research capacity building is essential for skilled workforce, technical competency and better construction management.

In order to scope with difficulties associated "poor geological condition", following remedial measure would be expected.

- Improvement of underground geological investigation (Exploratory Drilling, Geophysical Survey and Rock Mechanics Testing, etc.).
- Evaluation on rock mass classification along the tunnel by using proper forecasting methodology (Core Point Method, Indicator Krinking or Neutral Networks, etc.).
- Establishment of database system on tunnel specifications and method of statement based on past hydropower tunnels data in Myanmar.

For the mechanical factors, it is necessary for advancing geological investigation and estimation rock quality which can improve tunnel excavation and supporting system on weak geological area. Flanagan and Norman (1993) said that "Looking at the Past to forecast the Future" and presented a model of the forecasting process which any look into the future involves forecasting as illustrates in Figure 3.4. Forecasting is a non-mechanistic process which is not restricted to a purely mathematical evaluation of trends, such as the use of regression analysis, Box-Jenkins techniques or dynamic programming. To be of use, it requires breadth of vision and experience as well as competence in employing the forecasting methodology. It relies upon elucidating significant trends from past data. What forecasting tells us is that one way of dealing with such risk is to look at past experience of risk to infer the future riskiness of decisions.

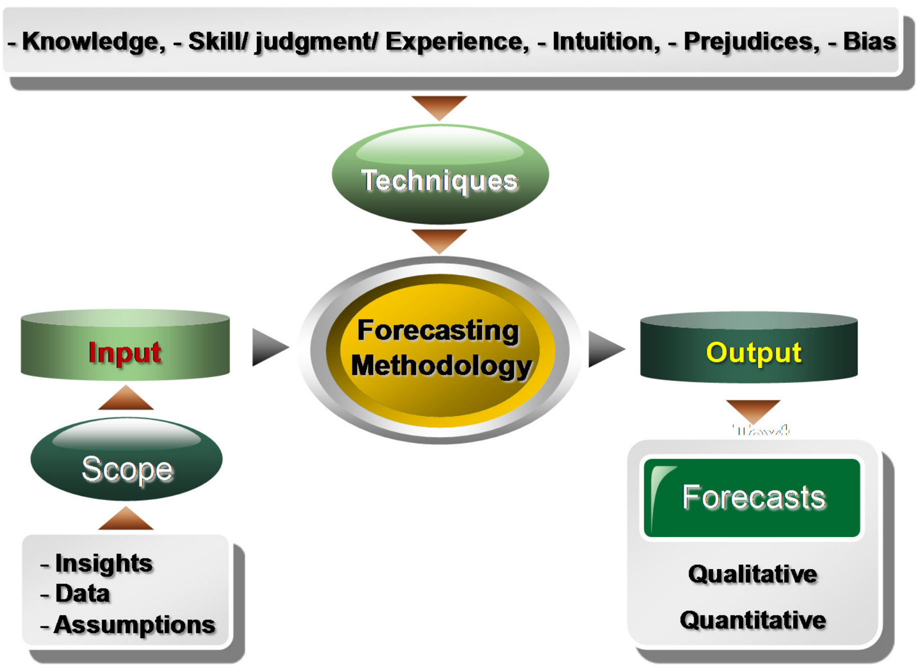


Figure 3.4: Forecasting Process (Source: Flanagan & Norman, 1993)

Whilst each forecasting has strengths and weaknesses, every situation is limited by constraints like time, funds or data. Some important considerations are:

- *Data availability*: It is pointless having a highly sophisticated technique if there is a lack of available data. The extent, accuracy and representativeness of the data are important.
- *Variability and consistency of the data*.
- *Time horizon*.
- *Cost of producing the forecast*.
- *Accuracy and reliability*.
- *Having an open mind about the future*.

To be of use, it requires breadth of vision and experience as well as competence in employing the forecasting methodology. It relies upon elucidating significant trends from past data.

Finally, it is done to have deeply understanding into the geo-risk management of human factors and mechanical factors on tunneling which make projects higher performance on geo-risk reduction in future.

3.5 Summary

Comparative study is the selected format for the study and it includes multiple projects which are from different geological conditions and implementation by different parties. Geo-risk reduction on mountain tunneling comprises risk identification through risk control on tunneling of hydropower projects by means of geo-risk management procedure. The composition of research methodology are presented as follows:

1) Description of risk management system.
2) Explanation of goal and scope of geo-risk management on tunneling.
3) Description on selection of study area which are most geo-risk effected on tunnel construction among the several hydropower projects.
4) Presentation of methodology on geo-risk management concerning with "Risk Identification", "Risk Classification,", "Risk Assessment" and "Risk Response" on tunneling of hydropower development.

5) Configuration of risk factors on tunneling such as human factors and mechanical factors, and figure out objective risk control for hydropower development. By improving human factors and mechanical factors would be recommended for mitigation of geo-risk on tunneling of hydropower projects in Myanmar.

CHAPTER 4

CASE STUDY OF GEO-RISK ON TUNNELING OF SITTAUNG VALLEY HYDROPOWER PROJECTS

4.1 General

4.1.1 Introduction

Generally, it is realized that hydropower construction has many uncertainties and risks. Among these, geological conditions of underground structures such as tunnel are one of the dominant risks, and it is essentially required a proper understanding of risks and their causes which can be efficiently manage and successfully mitigate cost and schedule risk. According to tunneling practice in hydropower projects, geological conditions of mountain tunnel are significant key to understand the behavior of unforeseeable geological conditions, "geo-risk". Therefore, the more understanding of related geological parameters through adequate exploration, in-situ surveys and evaluation of geology can help improving design patterns, preliminary cost estimation and risk mitigation of the hydropower tunnel.

4.1.2 Background of Hydropower Development

Myanmar is the second largest country in south-east Asia, bordering with five nations, Bangladesh, India, China, Laos, Thailand and is endowed with rich natural resources such as arable land, forestry, minerals and water resources. Now, continuing urbanization and industrial sectors are going to be the most important challenges. Electricity is essential among the infrastructures and industrial development, and one of the key indicators to measure the development of a nation. Myanmar is rich in hydropower potential and it can improve people living standard and industrial sectors. Among the Asian countries, present Myanmar's standard is presented in Figure 4.1 by means of human development index and electricity development of 2014. By this figure, it is clearly see that present Myanmar situation of human development index and lowest electric power usage among the Asian countries. Therefore, the role of hydropower will lead to the development of country in near future. However, the scales of hydropower projects are large and construction process is complicated, and "the cost and schedule risks on construction of hydropower projects" is main key risk factor which is effected by geotechnical failure behavior. Comparison of some projects cases study approach is conducted in order to figure out robust results of the study.

Ministry of Electric Power (MOEP) is state-owned and it acts as a main-contractor for hydropower development which has two major sectors one is power production and the other is power distribution. At present, the power production sector had been trying to implement large scale hydropower projects by Department of Hydropower Implementation (DHPI) to fulfill the electricity requirement of the country. Under the DHPI, ten numbers of Branches and seven numbers of Construction Divisions are carried out for the implementation of hydropower projects all over the country. The case study was conducted from two Construction Divisions of MOEP and one project is from local company. Following figures show development statue of Myanmar among the Asian Countries, history of electric power sector and organization chart of Ministry of Electric Power.

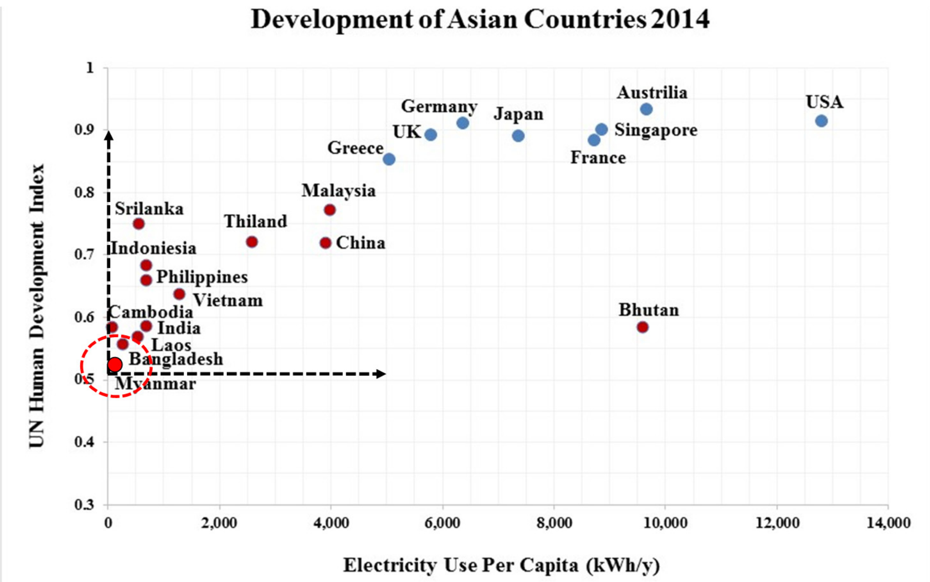


Figure 4.1: Electric Power Usage and Development of Asian Countries at 2014

(Source: UNDP Human Development Reports, 2014)

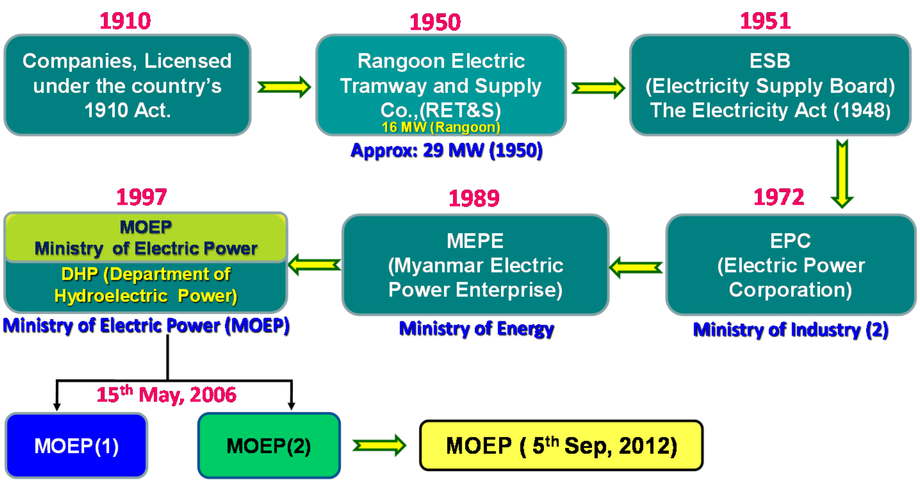


Figure 4.2: History of Electric Power Sector in Myanmar

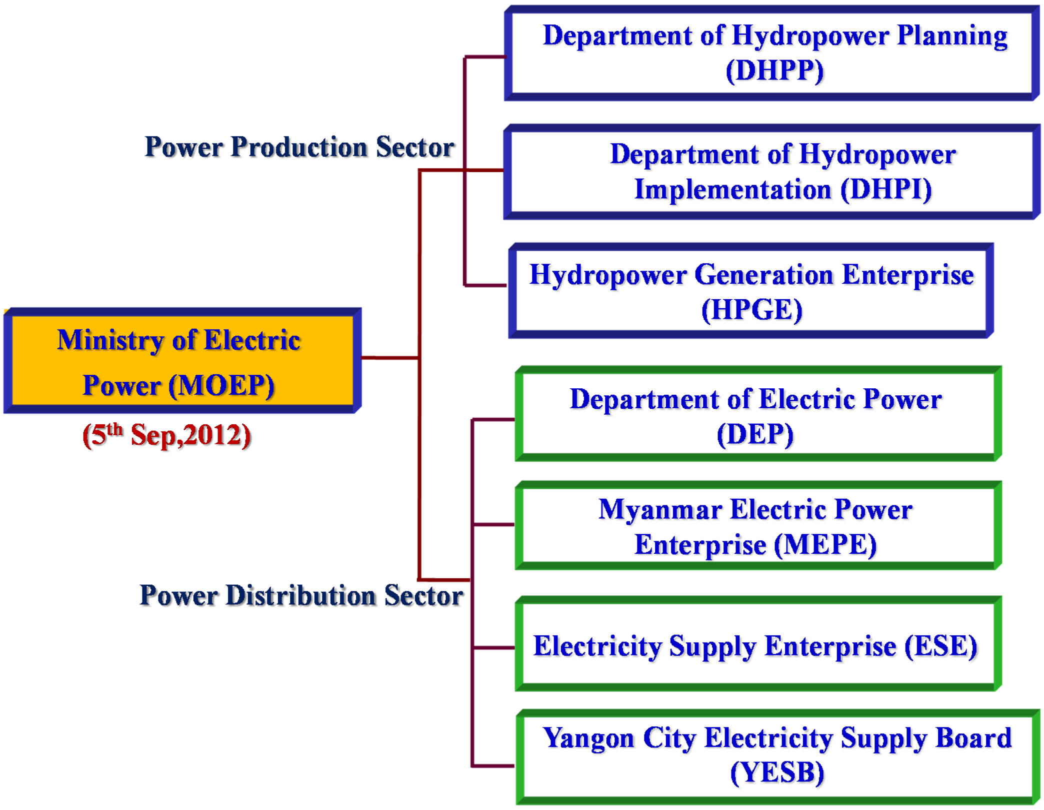


Figure 4.3: Organization of Ministry of Electric Power

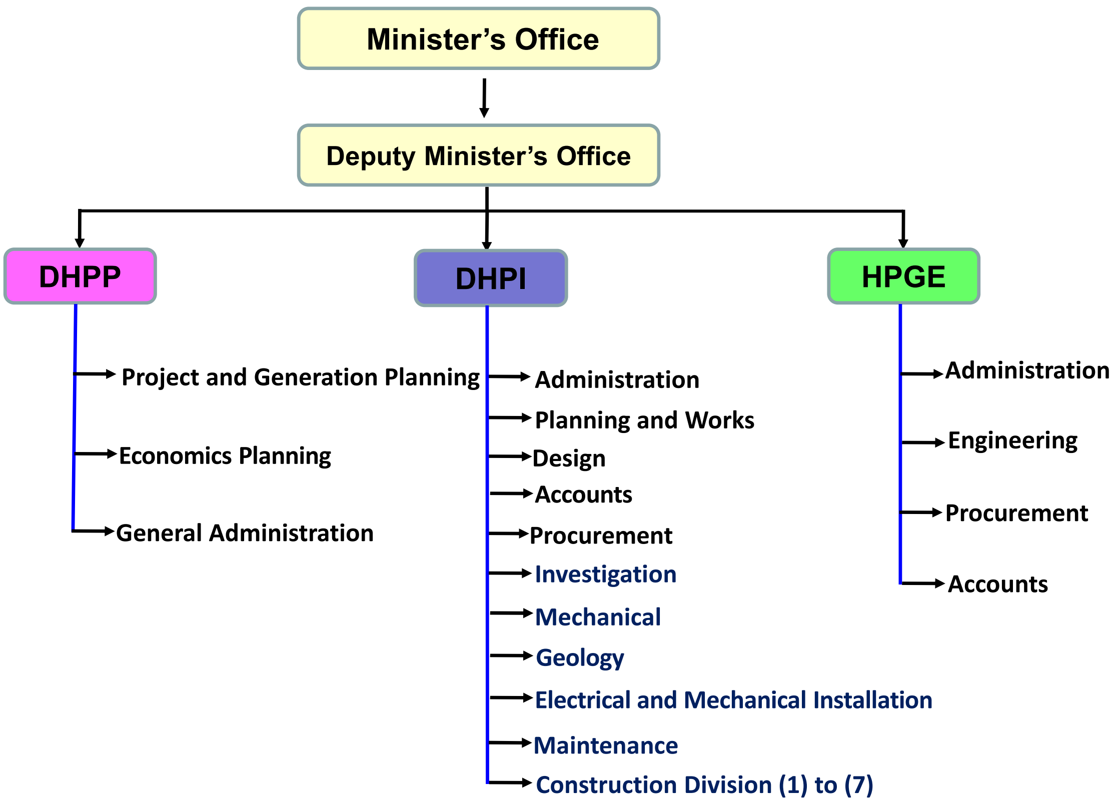


Figure 4.4: Hydropower Concerned Power Production Sector Organization Chart

4.1.3 Background of Case Study Projects

From the Sittaung valley hydropower projects, seven hydropower projects are selected for comparison of construction cost and schedule where two projects which is called Paunglaung and Nancho, are located in good geology and the other five projects which is called Kabaung, Phyu, Kun, Yenwe and Thaukyegat, are located in weak geology, and making comparison of tunnel excavation progress for four projects which are located in different geological conditions and similar tunnel structures. After that, geological assessment on two projects is carried out for understanding the geo-risk of tunnel excavation and figure out risk factors on construction. Both projects are located in weak and complex geological area. One is located on west to Sittaung river which is called Kun Hydropower Project by MOEP and the other is located on east to Sittaung river which is called Thaukyegat Hydropower Project by Local Company. Among the seven projects, five projects are facing with serious geological problem and encountering with tunnel failure cases during under construction, and the two projects have actual recorded rock mass classification data of tunnel excavated faces. Figure 4.5 presents the seven projects location and general impression of their geological situations within the Sittaung valley river basin.

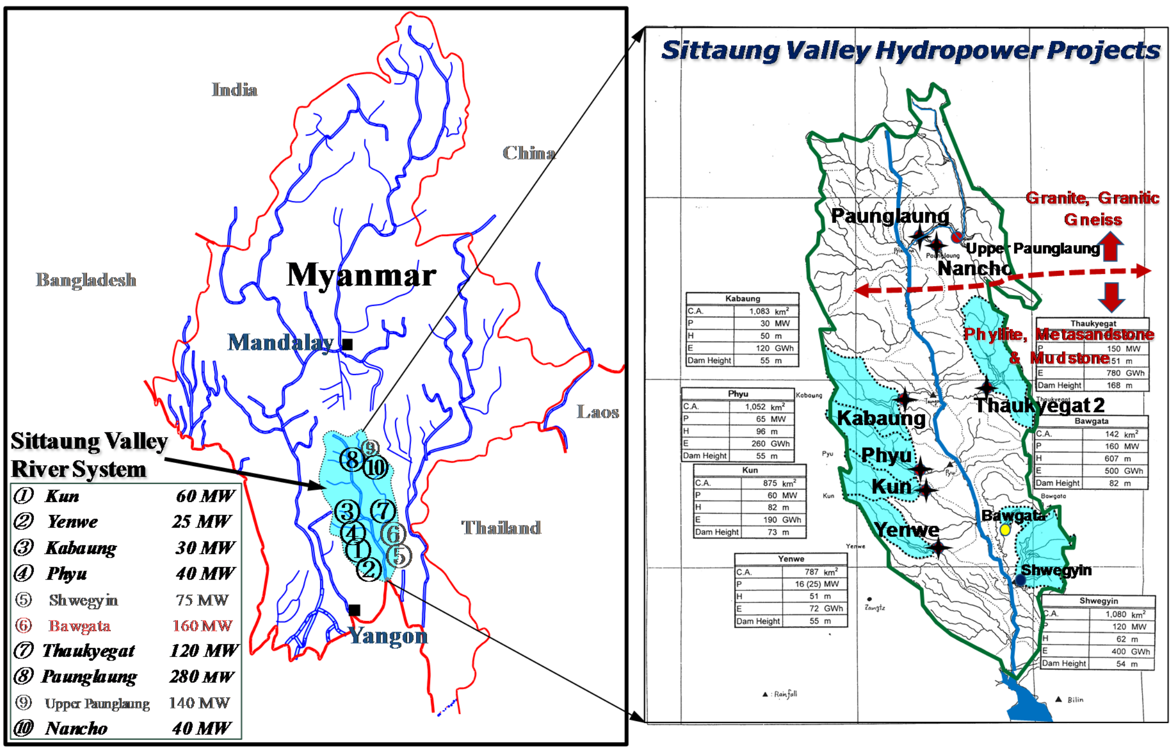


Figure 4.5: Study Area of Sittaung Valley River Basin (Source: KEPCO, 2008)

4.2 Overview of Geology

4.2.1 Geology and Topography of Myanmar

General description of geology and topography of Myanmar is as follows (KEPCO, 2002):

(1) Regional Geology

Myanmar can be subdivided into four geomorphic belts from east to west as follows:

a) Shan Highlands

Shan Highlands on the east part of Myanmar is characterized by the mountains with an average height of 1,200 m extending to N-S direction. The main components are consolidated, partially metamorphosed Paleozoic and Mesozoic rocks. Thanlwin River which flows from north to south in this belt is one of the largest rivers in this country. Limestone and its equivalents are widely distributed in this area.

b) Central Belt

Central belt is located on the mid lowland less than 300 m in altitude along Sittaung River and Ayeyarwaddy River and extends to the south part of Himalayan Mountains. This area is composed chiefly of Early Tertiary sediments and Cretaceous rocks.

c) Arakan-Chin Ranges

Arakan-Chin Ranges is located on the west area Ayeyarwaddy River and extends to the south part of Himalayan Mountains. This area is composed chiefly of Early Tertiary sediments and Cretaceous rock.

d) Arakan Coastal Plain

Arakan Coastal Plain is located on the west part of Myanmar from the coastal area toward to the inland. This area is formed of Tertiary sediments locally faulted and over thrust.

(2) *Tectonics*

The Tectonics of Myanmar is a result of the collision between the Indo Plate and the Asian Plate. The Indo Plate has drifted to the north contrary to the Asian Plate which has drifted to the south since the Tertiary. Accordingly, the collisions have affected the intensive faulting in this area.

(3) Faults

Major faults in Myanmar have two trend of strike. One is N-S direction and the other is NW-SE direction. Sagaing fault and Arakan Yoma fault represent the former type, which run in the western area. Pan Laung fault, Three Pagodas fault and Papun Wang Chao fault represent the later one, which run in the eastern area (Shan Highlands).

Sagaing fault is located on the boundary between Shan Highlands and Central belts. It is right-lateral fault due to the northward movement of Indo Plate.

(4) Seismogram

There are six (6) seismograms whose magnitude are more than 7.0 along Sagaing fault and one (1) whose magnitude is 8.0 along Taunggyi fault. Besides them, there are some records of more than 5.0 magnitude earthquakes.

Overview geology and topography of Myanmar figures are presented in Appendix A.

4.2.2 Regional Geological Condition of Study Area

Sittaung Valley Projects are located eastern part of BagoYoma where Kyaukkok formation of early Miocene age are exposed and Sino-Burma Ranges, which represent the largest tectonic unit in Myanmar. The Sino-Burma Ranges are bounded to the west with Sagaing Fault (Shan Boundary Fault) striking SSE-NNW. Its main components are consolidated, partly low-grade metamorphic, Paleozoic and Mesozoic sediments of the Burmese-Malayan Geosyincline (KEPCO, 2007).

The physiographic unit of Sittaung valley is generally subdivided into three (3) geological units such as east Kachin/ Shan Unit, west Kachin unit and Karen/ Tenasserium Unit. This Unit is bounded by Sagaing Fault (Winse, 1972) in the west and Nwalabo-Paunglaung Fault Belt (Garson et al., 1976) in the east. The Sagaing Fault is a regional strike-slip fault with more than 1,000 kilometer long right lateral movement that is active from Miocene to Recent (Mitchell, 1993) as illustrated in Figure 4.6.

Along the eastern flank of the Bago Mountains, Sagaing Fault runs in a north-south direction. The Fault is a right-lateral strike-slip transform fault bordering between the Burma Plate (Micro-plate) and the Shan Plate (Micro-plate). The Bago Mountains were strongly folded

by the post Himalayan orogenic movement and has also affected by the movement of the Sagaing Fault.

Geology condition of projects area is different between east side and west side of Sittaung River, and quality of rock is also quite different between upper most of valley and rest of lower valley. At west side of Sittaung River on lower valley, the rocks are predominantly made up of fine to medium grained, medium hard sandstones inter bedded with minor shale and siltstone. At east side of Sittaung River on lower valley, the rocks are consisting of phyllite, quartzite and schist are distributed together with granites which sandwiches the metamorphic rocks.

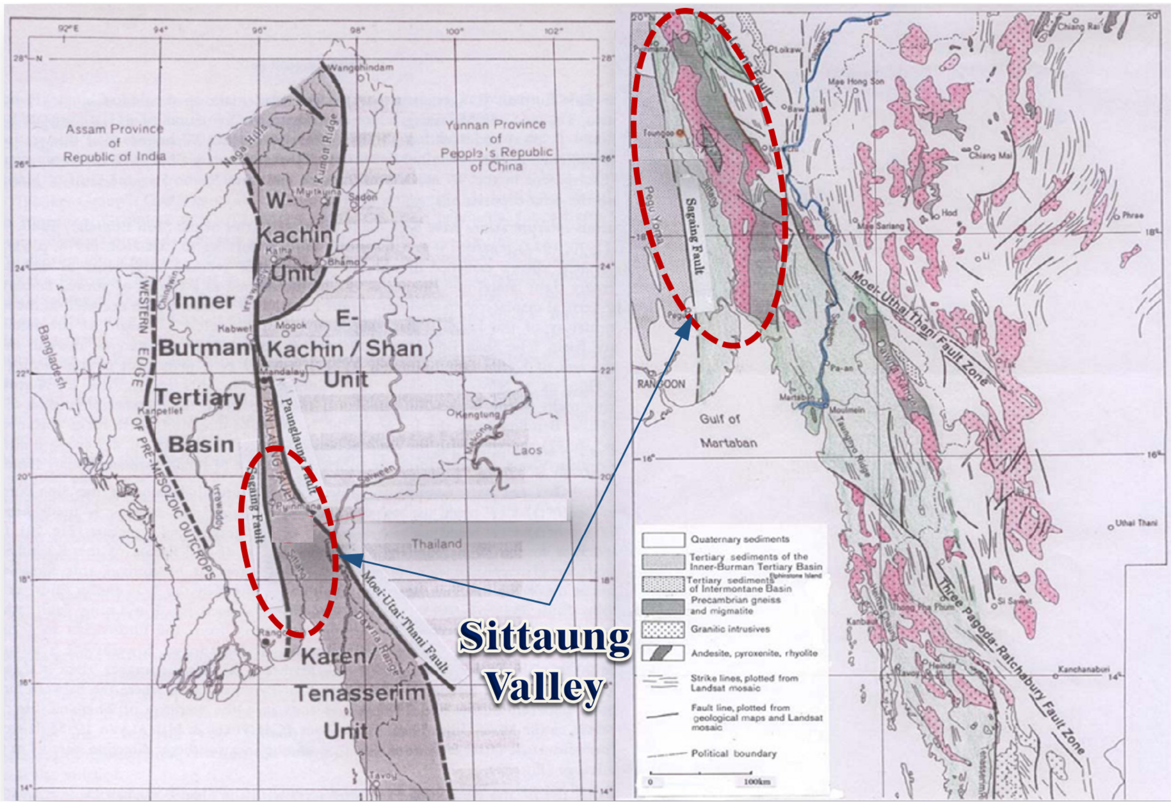


Figure 4.6: Tectonic Domains and Sketch Map of Sittaung Valley (Geology of Burma 1983) (Source: KEPCO, 2007)

4.3 Comparative Study on Tunneling Progress

4.3.1 Introduction

Kabaung, Phyu, Kun and Yenwe projects are implemented by Construction Division No.3, and Paunglaung and Nancho projects are implemented by Construction Division No.1 under the DHPI. Thaukyegat project is implemented by Local Company. Firstly make a

comparative study on progress of tunnel excavation of four projects which are having similar tunnel structures and located in different geological conditions namely Kun, Thaukyegat, Paunglaung and Nancho. Kun and Nancho project have similar tunnel structure but Kun is located in complex geological area and Nancho is located in good geological area. Thaukyegat and Paunglaung project also have similar tunnel structure but Thaukyegat is located in complex geological area and Paunglaung is located in good geological area. Comparative study is carried out by means of these two pair of four projects. Figure 4.7 shows the locations of four projects.

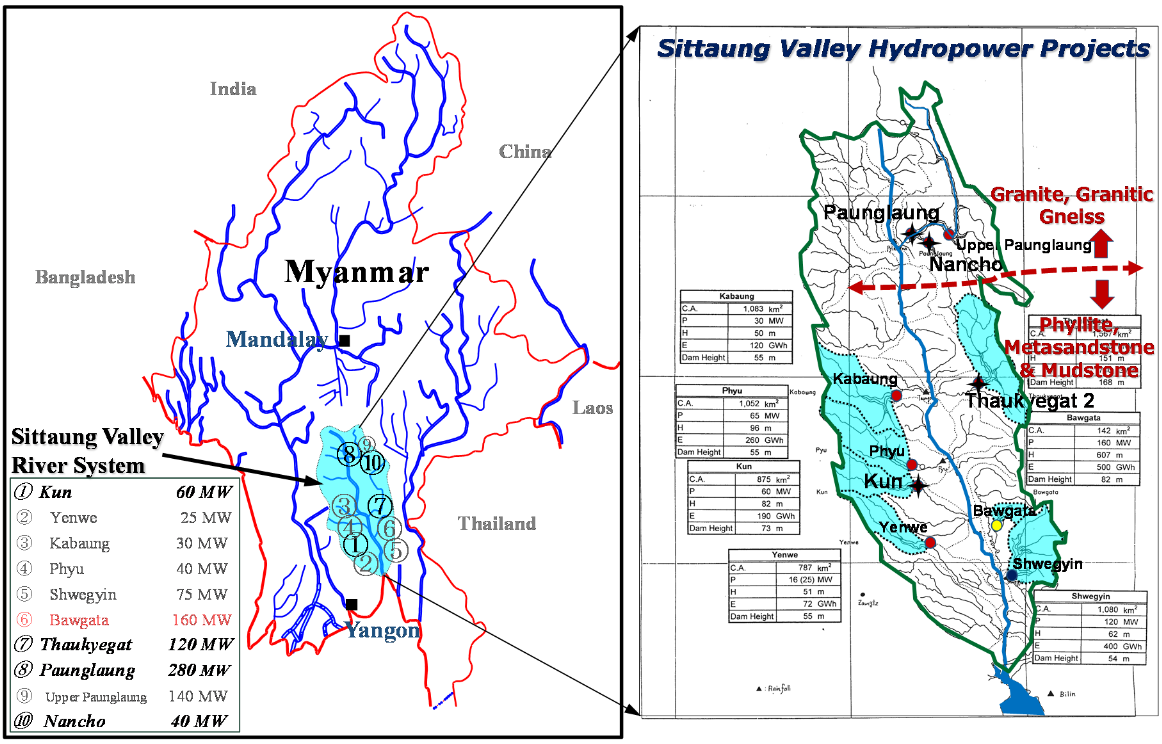


Figure 4.7: Location Map of the Four Projects Study Area (Source: KEPCO, 2008)

4.3.2 Geology Background and Features of the Four Projects

Kun Project is located on Kun river, a tributary from the west to Sittaung river. Kun river runs through steep gorges with many rapids in the upper reaches and through alluvial plains in lower reaches before it merges with Sittaung river.

Regional Geology of Kun project indicates that the rocks are predominantly made up sandstone inter bedded with minor shale and siltstone in the project area.

Thaukyegat Project is located on Thaukyegat river, a tributary from the east to Sittaung River. Thaukyegat river originated in higher mountains and sharply drops and then gradually

lower elevation to the confluence which is about 120 km long from the source of the river to the confluence with the Sittang river.

Regional Geology of Thaukyegat project indicates that Paleozoic metamorphic rocks consisting of phyllite, quartzite and schist are distributed in the dam site together with granites which sandwiches the metamorphic rocks after all.

Nancho Project is located on Nancho river, one of east tributaries of the Paunglaung river, it runs northward repeating small meanders with an elongated watershed, situated in the mountainous region of the western edge of the Shan Plataue between Paunglaung Fault in the east and Sagaing Fault in the west.

Regional Geology of Nancho project indicates that this area granite intrusion took place in the Mesozoic to Cenozoic Era. The bedrocks of the Project site consist of gneiss, schist, meta-sandstone and gneissose granite.

Paunglaung Project is located on Paunglaung river, a tributary from the east to Sittaung river. River valley is narrowed with steep slope on both sides and widens out for a few kilometers from dam site before it enters the Sittaung valley.

Regional Geology of Paunglaung project indicates that basin is situated in the Sino-Burma Ranges, the eastern-most physiographic unit in Myanmar, where geologically older rocks are widely distributed. The bedrocks of the Project site consist of gneiss, schist, meta-sandstone and gneissose granite.

Figure 4.8 illustrates the projects layout and salient features of the projects are tabulated in Table 4.1

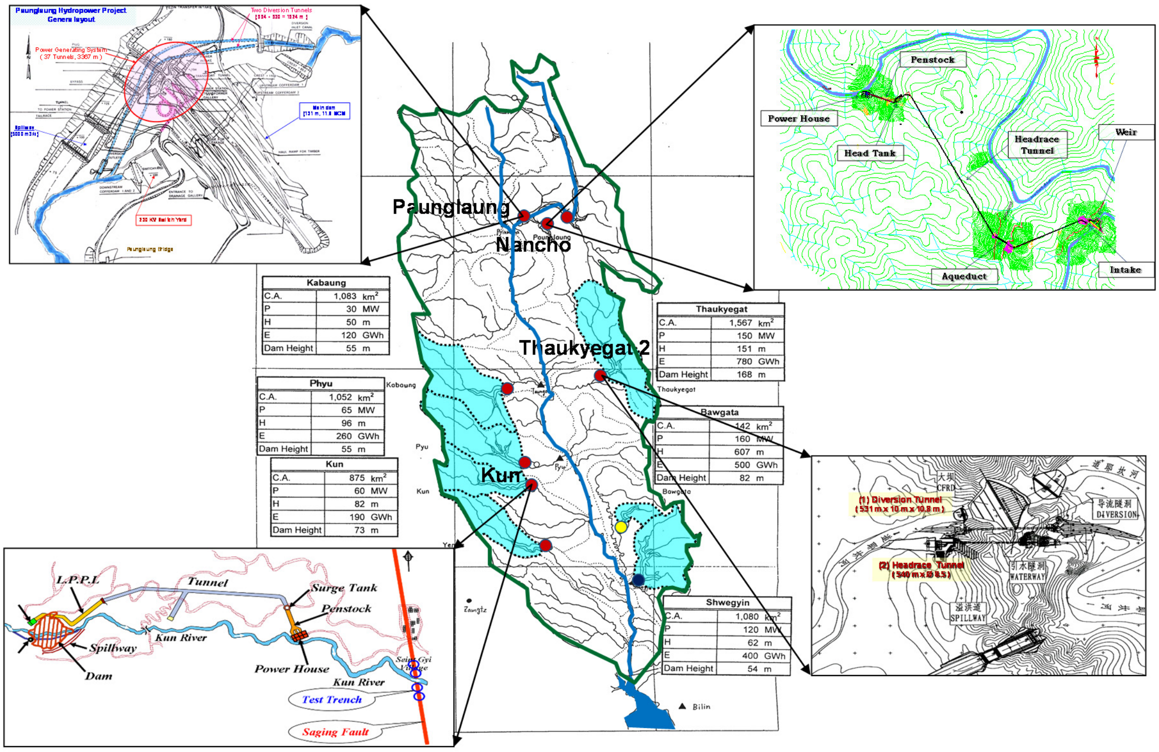


Figure 4.8: Location Map of Study Area of the Projects

Table 4.1: Salient Features of Kun, Nancho, Thaunyegat and Paunglaung Project

Sr. No.	Particulars	Projects			
		Kun	Nancho	Thaukyegat	Paunglaung
I	**Power Indices**				
1	**Installed capacity (MW)**	**60**	**40**	**120**	**280**
2	Plant discharge at rated head (m^3/s)	66	47	70	75
3	Design head	106.0	95.7	64.5	103.6
4	**Energy Generation (GWh)**	**190**	**152**	**605**	**911**
II	**Hydrology and Reservoir**				
1	Catchment aera (km^2)	875	821	2152	4381
3	Annual mean inflow (Mm^3)	741	946	4220	4040
III	**Dam**	Earth Core Rock Fill Dam (**2.24 Mm^3**)	Gravity Dam (**0.068 Mm^3**)	Concrete Faced Rock Fill Dam (**2.67 Mm^3**)	Earth Core Rock Fill Dam (**11.80 Mm^3**)
IV	**Diversion Tunnel**	Rectangular Conduit	Rectangular Conduit	Tunnel (Horseshoe)	Tunnel (Horseshoe)
1	**Length (m)**	**345**	**140**	**531**	**994**
2	**Diameter/ W & H (m)**	**1.5 x 3.8 x 3**	**2.5 x 3.75**	**11 x 13**	**10 x 14**
V	**Spillway**	Ungated Spillway	Ungated Steped Spillway	Gated Spillway	Ungated Steped Spillway
VI	**Power Intake**	Tower with Control Gate	Gated Type	Gated Shaft Type	Inclined Tower with Control Gate
VII	**Headrace Tunnel**	Low Pressure Type Circular Shape	Non-pressure Type Horse-shoe Shape	Pressure Type Circular Shape	Pressure Type Circular Shape
1	**Length x Diameter (m)**	**1755 x 5.5**	**2352 x 4.72**	**538 x 8.5**	**80 x 8.4**
VIII	**Penstock**	Above Ground Type	Above Ground Type	Underground Ground Type	Underground Ground Type
IX	**Powerhouse**	Semi-undrground Type	Semi-undrground Type	Semi-undrground Type	Undrground Type

4.3.3 Comparison on Tunneling Progress of the Four Projects

For the First Pare Projects of Kun and Nancho

Kun Project

As aforesaid in above table, it is notified that the Kun project is Dam and waterway type with medium scale power plant. In addition, the "dam and waterway type" development was much preferable than dam type to increase both installed and firm capacities by means of technically and economically feasible. It is having the installed capacity of 60 MW by use of the rated gross head of 106 m and the maximum discharge of 66 m^3/s.

As for the study, in the headrace, there is two type of low pressure structures which one is low pressure pipe line and the other is low pressure tunnel. The headrace tunnel have a total length of approximately 1,755 m, consisting of four work faces which are inlet portion, outlet portion, and the other two are upstream and downstream from the adit tunnel. The headrace tunnel is low pressure type with concrete lining, and circular shape with a diameter of 5.5 m and the gradient of 1/400 ~ 1/500. Project layout is presented in Figure 4.9.

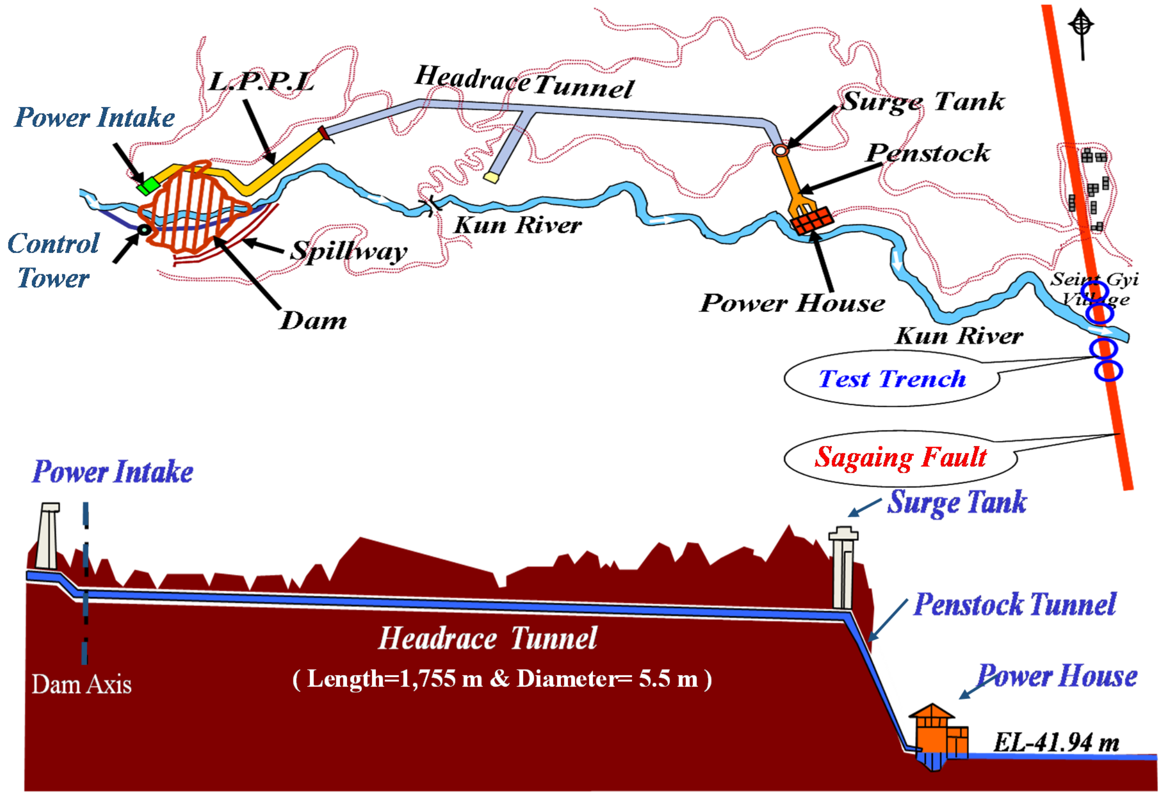


Figure 4.9: General Layout of Kun Hydropower Project (Source: KEPCO, 2008)

Nancho Project

As mention in above table, it is notified that the Nancho project is also Dam and waterway type with medium scale power plant. It was revealed that the "run-of-river type (waterway type)" development has better economy than the "reservoir type" development through the provisional study. Finally, it was adopted as the "dam and waterway type" to increase both installed and firm capacities at the reasonable sacrifice of the economy and having the installed capacity of 39.7 MW by use of the rated gross head of 100 m and the maximum discharge of 47.4 m^3/s.

As for the study, the headrace tunnel have a total length of approximately 2,352 m, consisting of an upstream tunnel from the power intake to the Salu river right bank and a downstream tunnel from the Salu river left bank to the head tank. It is non-pressure type with concrete lining. The shape of the tunnel after lining is horse shoe type with a nominal height and width of 4.72 m and the gradient of 1/1,000. Project layout is presented in Figure 4.10.

The main point of comparative study on different geology and similarity of structures of the projects are tabulated in Table 4.2 and their tunnel profiles are illustrated in Figure 4.11.

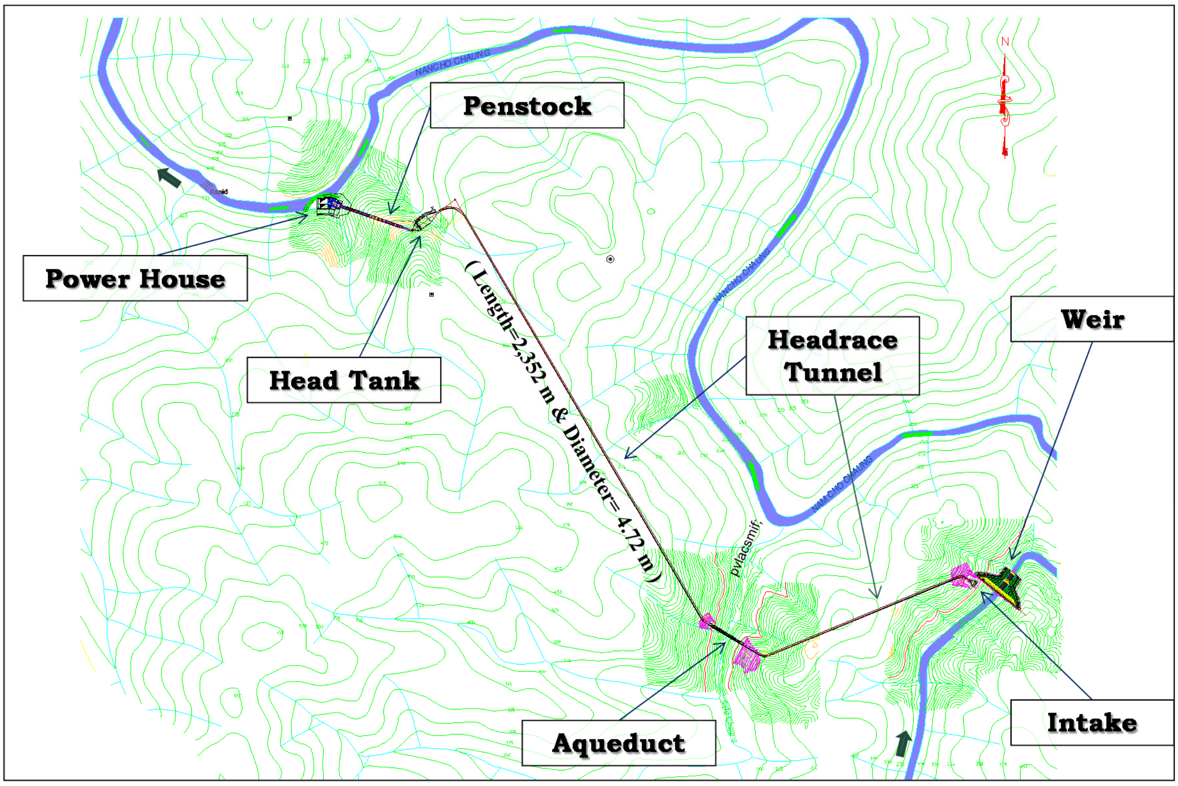


Figure 4.10: General Layout of Nancho Hydropower Project (Source: KEPCO, 2008)

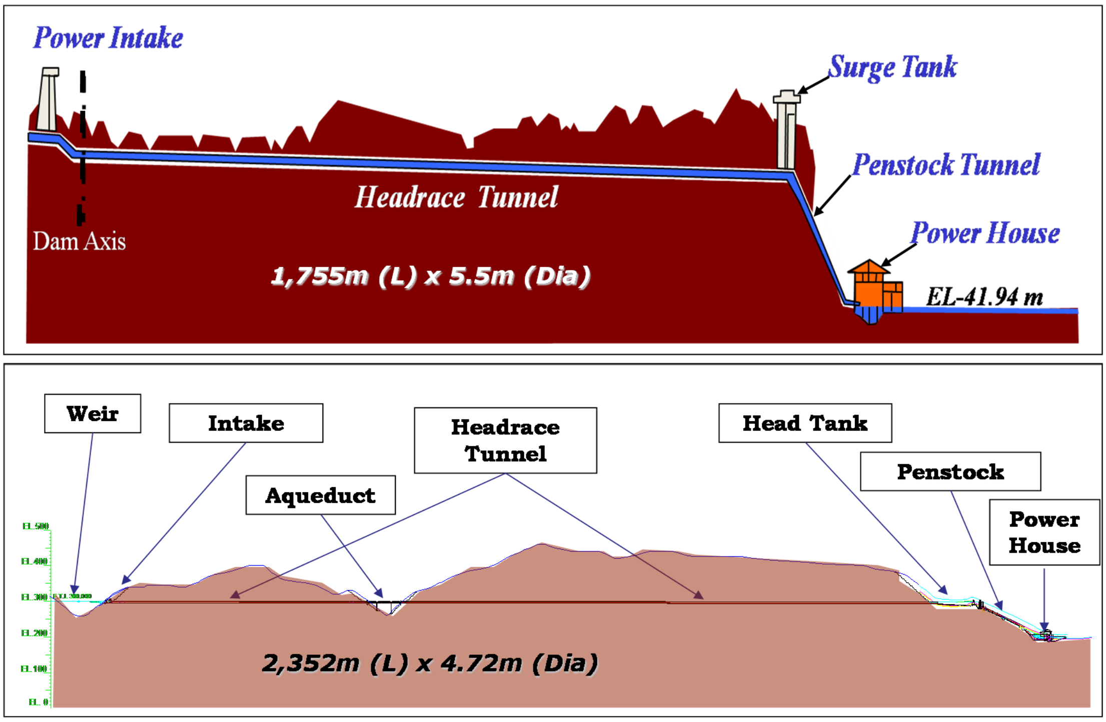


Figure 4.11: Waterway Profile of Kun and Nancho Project (Source: KEPCO, 2008)

Table 4.2: Different Geology and Similar Structure of Kun and Nancho

Situation	KUN	Nancho
1. Location		
(1) Sittaung Vally	Downstream most & West to Sittaung River	Upstream most & East to Sittaung River
2. Geological Condition		
(1) Lithology	Meta-sandstone, Mudstone (weak)	Granite, Granitic Gneiss (good)
3. Structure		
(1) Diversion Conduit (W x H)	1.5 x 3.8 m	2.5 x 3.75 m
(2) Headrace Tunnel (L x Diameter)	1755 x 5.5 m	2352 x 4.72 m
4. Power Indices		
(1) Installed Capacity (MW)	60	40
5. Organization		
(1) Implementation by	Construction Division No.3 (MOEP)	Construction Division No.1 (MOEP)

For the Kun project, the progress of tunnel excavation on penstock portion where is existed in shear mud stone zone was selected for the progress study. Excavation was take time about 3.7 months for 102 m long which progress was 26 m for monthly and maximum exploration was 30 m per month under the condition of weak geology. The progress of tunnel excavation

was taken time about 7.5 years and concrete lining for 4 years. The overall construction period for headrace tunnel was long last for 8 years. The progress of the penstock tunnel is presented in Figure 4.12.

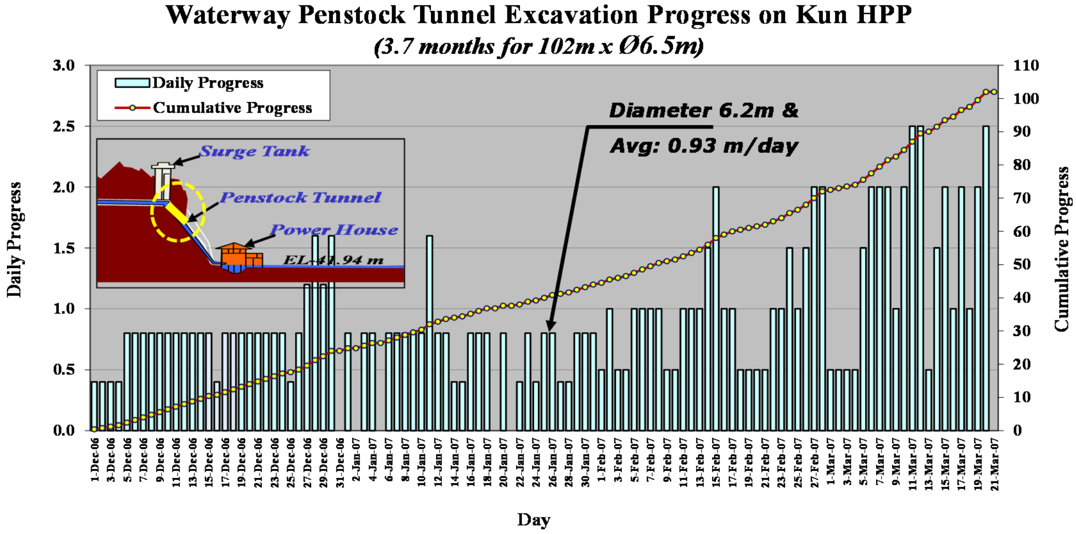


Figure 4.12: Penstock Tunnel Progress of Kun Project

For the Nancho project, the headrace tunnel excavation was taken time about 10 months for 711 m long which progress was 39 m for monthly and maximum exploration was 113 m per month under the condition of better geology. The progress of tunnel excavation was taken time about 3.35 years and concrete lining for 3.75 years. The overall construction period for headrace tunnel was long last for 5 years. The progress of the headrace tunnel is presented in Figure 4.13.

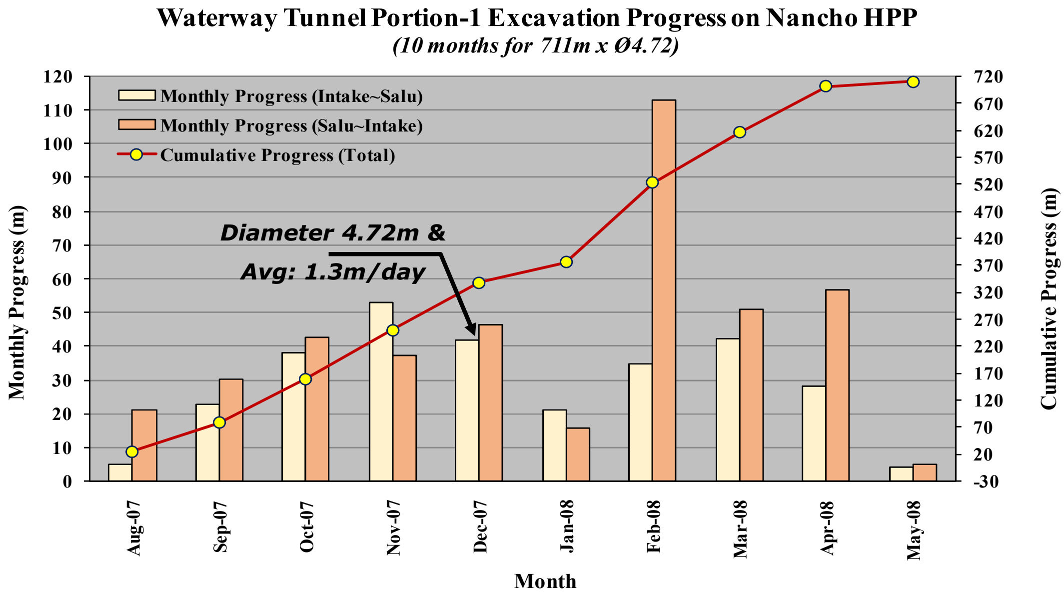


Figure 4.13: Headrace Tunnel Progress of Nancho Project

Figure 4.14 illustrates for comparative study on tunneling of two projects. Penstock tunnel of Kun project was not able to be much speedy because of weak geology (sheared mudstone zone) and progress is about 26 m per month and maximum excavation speed is about 30 m per month. Waterway tunnel of Nancho project was can speedy because of good geology (Granite, Granitic Gneiss) and progress is about 39 m per month and maximum excavation speed is 113 m per month.

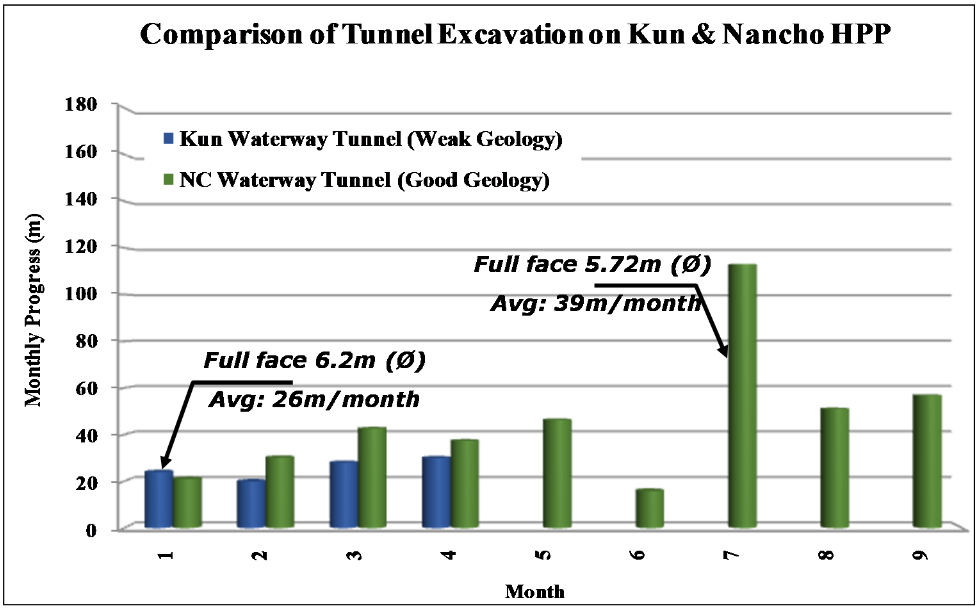


Figure 4.14: Comparison of Tunnel Excavation on Kun and Nancho HPP

For the Second Pare Projects of Thaukyegat and Paunglaung

Thaukyegat Project

As aforesaid in Table 4.1, it is notified that the Thaukyegat project is Dam type with large scale power plant. The "dam type" development was much preferable from the viewpoint of technical and economical solution. It is having the installed capacity of 120 MW by use of the rated gross head of 64.5 m and the maximum discharge of 70 m^3/s.

As for the study, there is two tunnel structures which one is diversion tunnel and the other is waterway tunnel. The diversion tunnel have a total length of approximately 531 m, consisting of two work faces which are inlet portion and outlet portion. The tunnel is pressure type with concrete lining, and horse shoe type with a size of 11 m x 13 m. The waterway tunnel will have a total length of approximately 538 m, consisting of four work faces which are inlet, outlet, and adit tunnel upstream and downstream. The headrace tunnel is low

pressure type with concrete lining, and circular type with a diameter of 8.5 m. Project layout is presented in Figure 4.15.

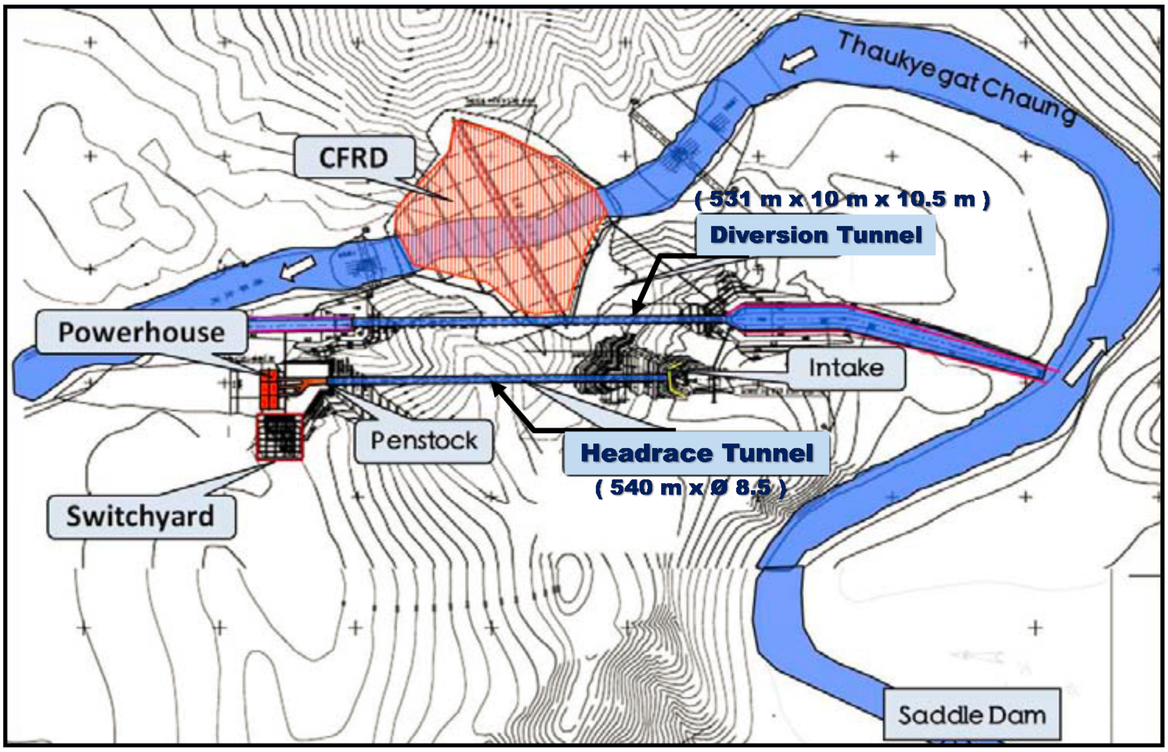


Figure 4.15: General Layout of Thaukyegat Hydropower Project (Source: DHPI, 2012)

Paunglaung Project

As mention in Table 4.1, it is notified that the Paunglaung project is Dam type with large scale power plant. This project was most suited for underground power structures from the better opportunity of existing foundation geology. It is having the installed capacity of 280 MW by use of the rated gross head of 103.6 m and the maximum discharge of 75 m^3/s. As for the study, the two diversion tunnels have each total length of about 994 m and 930 m. It is pressure type with concrete lining. The tunnels are pressure type with concrete lining, and horse shoe type with a size of 10 m x 14 m. Project layout is presented in Figure 4.16. The main point of comparative study on different geology and similarity of structures of the projects are tabulated in Table 4.3 and their tunnel profiles are illustrated in Figure 4.17.

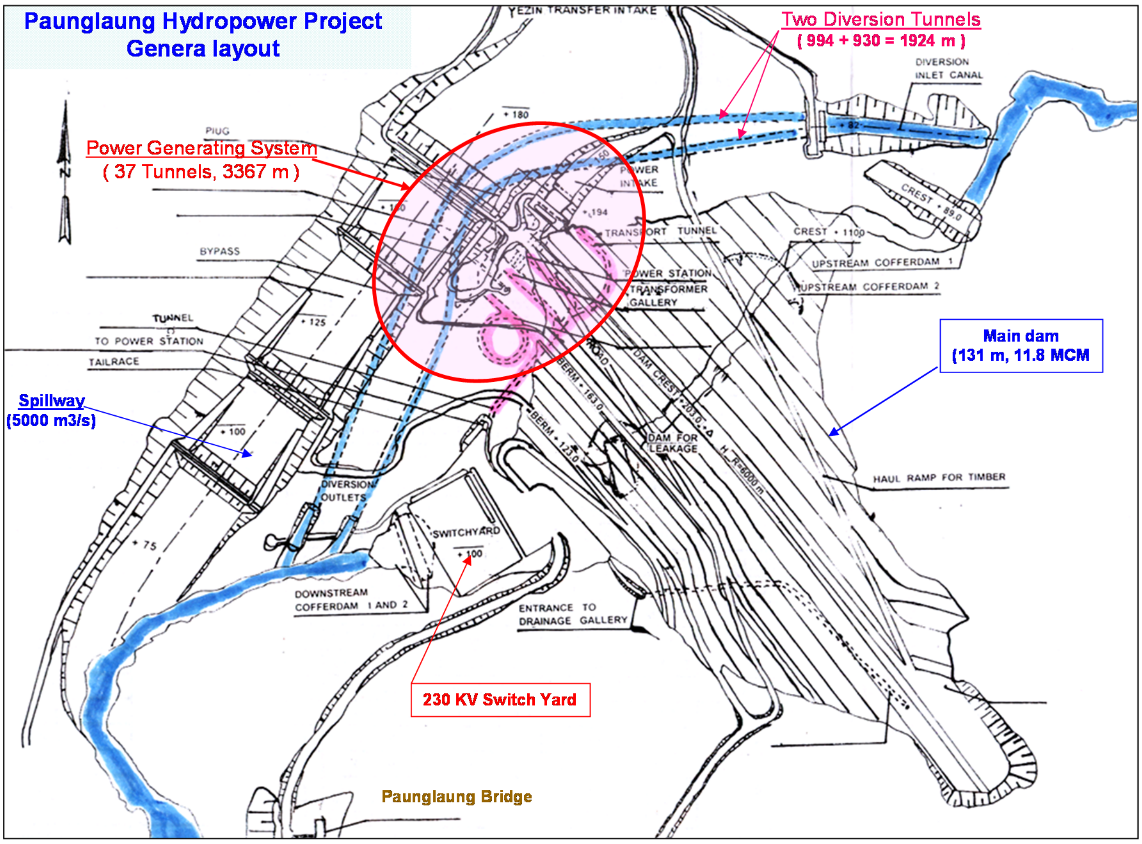


Figure 4.16: General Layout of Paunglaung Hydropower Project (Source: Wan Kyi, 2009)

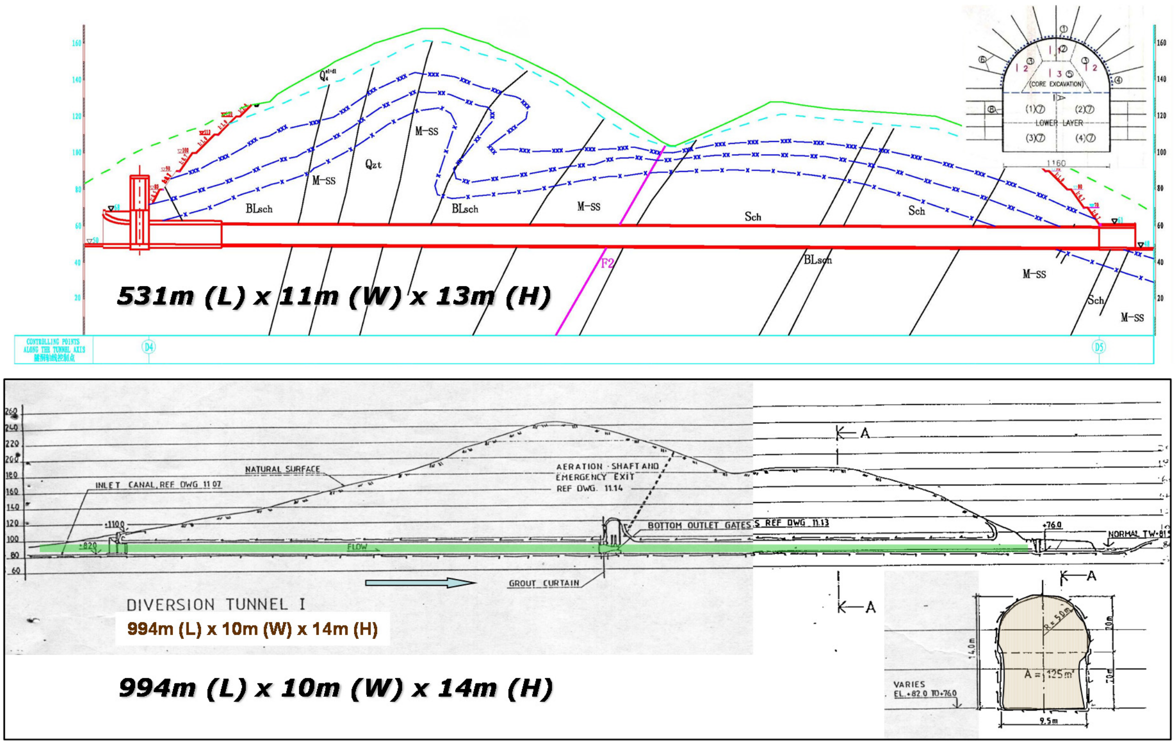


Figure 4.17: Diversion Profile of Thaukyegat Project and Paunglaung Project

(Source: Wan Kyi, 2009)

Table 4.3: Different Geology and Similar Structure of Thaukyegat and Paunglaung

Situation	Thaukyegat	Paunglaung
1. Location		
(1) Sittaung Vally	*Middle Downstream & East to Sittaung River*	*Upstream most & East to Sittaung River*
2. Geological Condition		
(1) Lithology	*Phyllite, Schist, Meta-sandstone, (weak)*	*Granite, Granitic Gneiss (good)*
3. Structure		
(1) Diversion Tunnel (L x W x H)	*531 x 11 x 13 m*	*994 x 10 x 14 m*
(2) Headrace Tunnel (L x Diameter)	*538 x 8.5 m*	*80 x 8.5 m*
4. Power Indices		
(1) Installed Capacity (MW)	***120***	*280*
5. Organization		
(1) Implementation by	*Gold Energy Co., Ltd (Local Company)*	*Construction Division No.1 (MOEP)*

For the Thaukyegat project, the progress of diversion tunnel excavation was taken time about 18 months for 531 m long which progress was 45 m for monthly and maximum exploration was 86 m per month under the condition of highly to moderate weathered rocks. The progress concrete lining was 10 months. The overall construction period for diversion tunnel was long last for 2.2 years. And then, the progress of waterway tunnel excavation was taken time about 8 months for 538 m long which progress was 108 m for monthly and maximum exploration was 140 m per month under the condition of weak geology. The progress of concrete lining was 14 months. The overall construction period for waterway tunnel was long last for 1.75 years.

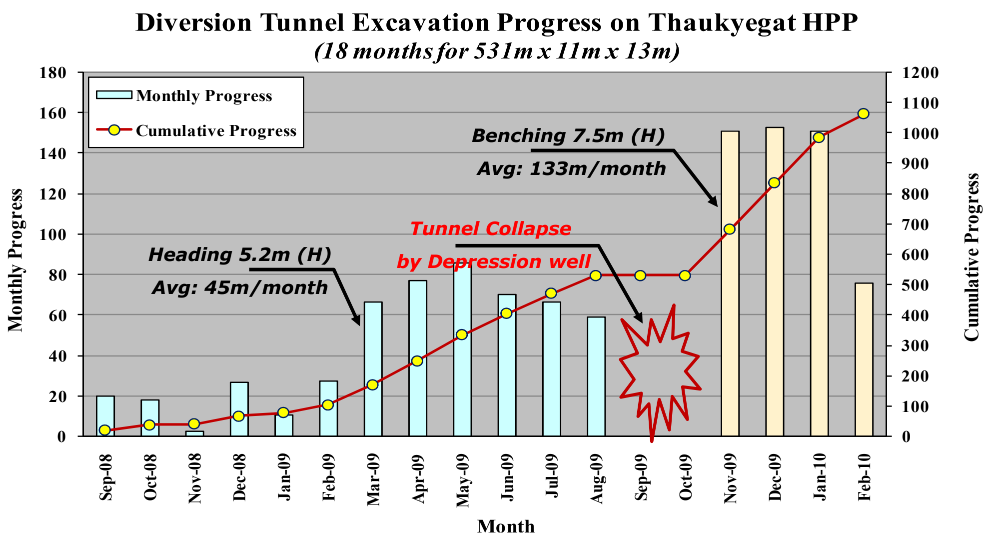


Figure 4.18: Diversion Tunnel Progress of Thaukyegat Project

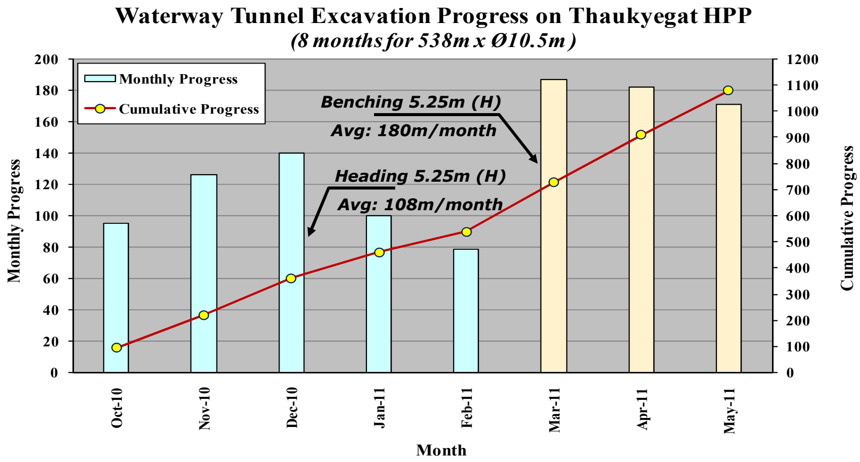


Figure 4.19: Waterway Tunnel Progress of Thaukyegat Project

For the Paunglaung project, the progress of diversion tunnel No.1 excavation was taken time about 20 months for 994 m long which progress was110 m for monthly and maximum exploration was 283 m per month under the condition of good geology. The progress of concrete lining was 11 months. And then, the progress of diversion tunnel No.2 excavation was taken time about 26 months for 930 m long which progress was 62 m for monthly and maximum exploration was 180 m per month under the condition of good geology. The progress of concrete lining was 1 year. The progress concrete lining was 10 months. The overall construction period for two diversion tunnels were long last for 2.9 years. The progress of two diversion tunnels is presented in Figure 4.20.

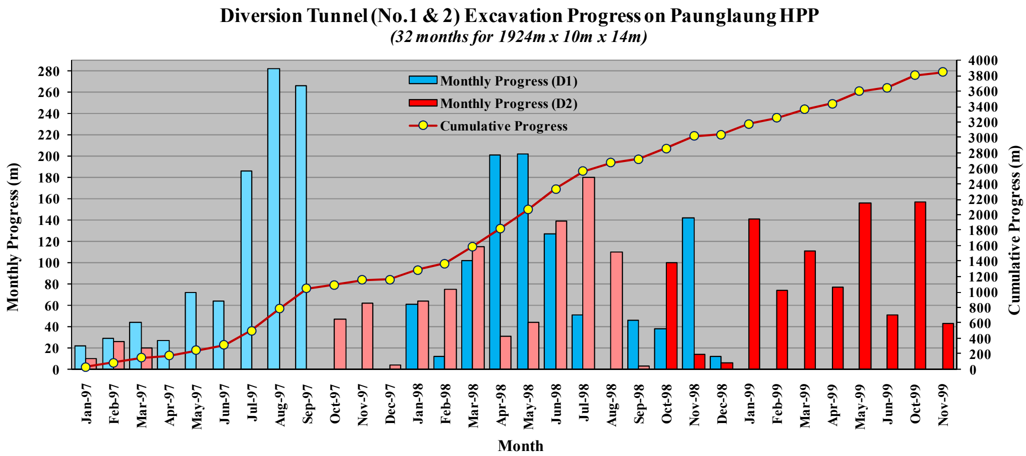


Figure 4.20: Two Diversion Tunnels Progress of Paunglaung Project

Figure 4.21 illustrates for comparative study on tunneling of two projects. Diversion tunnel of Thaukyegat project is not able to be much speedy because of weak geology (highly weathered rock with ground water) and progress is about 45 m per month and maximum excavation speed is about 86 m per month. However, diversion tunnels of Paunglaung

project were can speedy because of good geology (Granite, Granitic Gneiss) and progress is about 110 m per month and maximum excavation speed is 283 m per month.

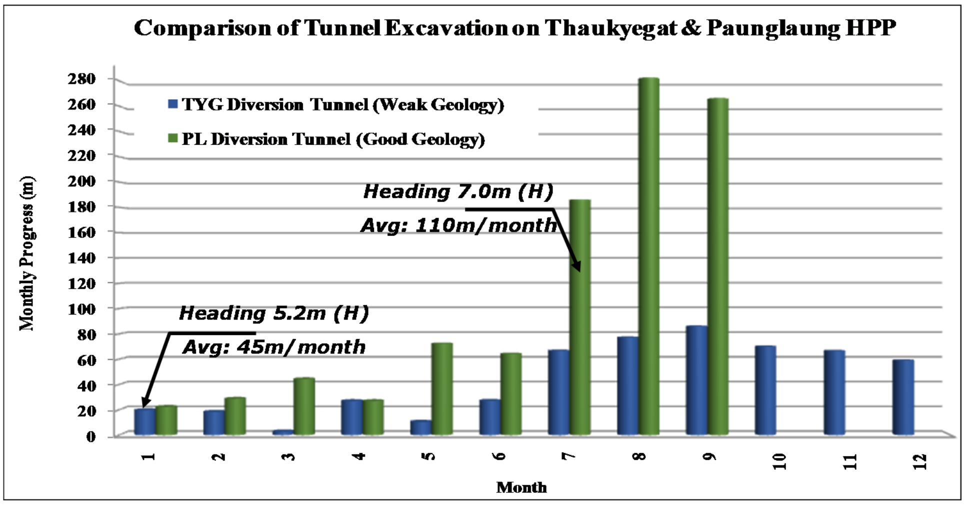


Figure 4.21: Comparison of Tunnel Excavation on Thaukyegat Project and Paunglaung Project

Figure 4.22 is presented the comparison of four projects tunnel works which are implemented by different parties and located in different geological conditions. It is noticed that all of the tunnel excavation works cannot much speedy on early stage, and inlet and outlet area of the mountain. However, after inlet and outlet area, tunneling works can speedy on both weak and good geological conditions of the inside of the mountain. It can be clearly see that the tunnel in the better geological area can explore more progress than weak geology and systematic geological observation is essential.

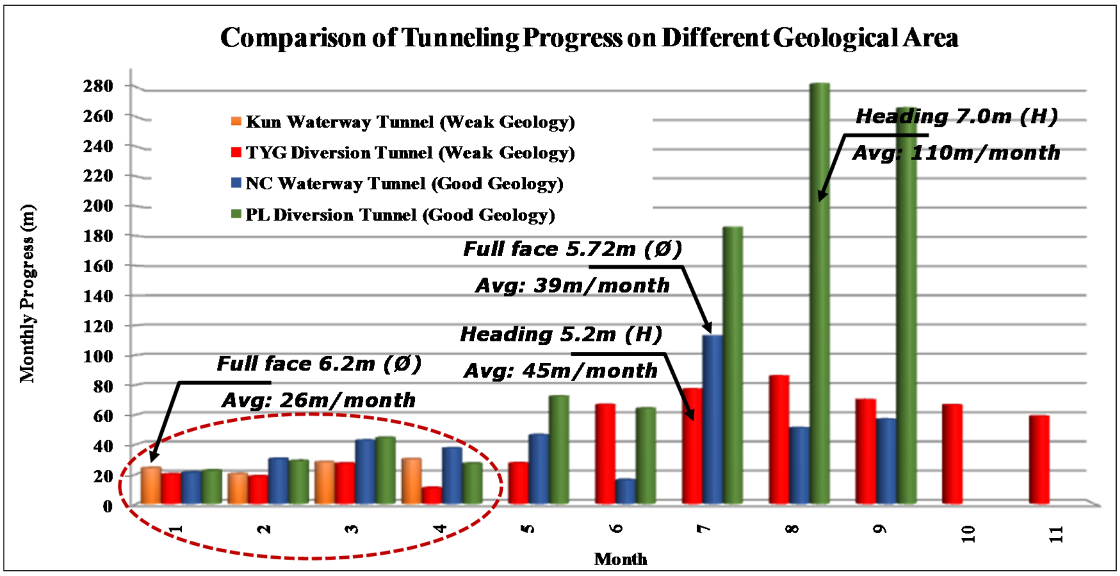


Figure 4.22: Comparison of Tunnel Excavation on Sittaung Valley Projects

4.3.4 Review on Tunneling of the Four Projects

Based on above study, it is noticed that tunneling in the region of good geology are simple and poor construction does not much effect on tunneling, but tunnel construction in poor geology face much complicated disturbances leading to collapse and poor construction also heavily effect on tunneling. Table 4.4 and 4.5 present the comparison of evaluation on cost and schedule risk by geo-risk on tunneling of four hydropower project. To overcome cost and schedule risks for hydropower development, systematic geological observation, procurement of tunnel tools and equipment are essential for tunnel excavation on every kind of geological conditions.

Table 4.4: Comparison on Projects Scale and Geological Conditions

No.	Projects	Power		Geology Condition	Remarks
		Installed (MW)	Energy (Gwh)		
1	Kun	60	190	*Metasandstone & Mudstone (weak)*	*By MOEP Con; Period: 2002~2012*
2	Thaukyegat	120	605	*Phyllite, Schist & Metasandstone (weak)*	*By Local Company Con; Period: 2008~2013*
3	Nancho	40	152	*Granite & Granitic Gneiss (good)*	*By MOEP Con; Period: 2005~2014*
4	Paunglaung	280	911	*Granite & Granitic Gneiss (good)*	*By MOEP Con; Period: 1997~2005*

Table 4.5: Comparison on Cost and Schedule of the Projects

No.	Projects	Construction Period (years)			Project Cost (MUSD)			Remarks
		Planning	Actual	Delay	Initial	Final	Over (%)	
1	Kun	6	11	**5**	63.22	108.45	**72**	*1MW cost is 1.8 M$*
2	Thaukyegat	3.5	5	**1.5**	234.94	249.62	**6**	*1MW cost is 2.1 M$*
3	Nancho	5	9	**4**	41.31	60.10	**45**	*1MW cost is 1.5 M$*
4	Paunglaung	5.5	8	**2.5**	369.67	371.16	**0.4**	*1MW cost is 1.3 M$*

4.4 Geological Assessment on Tunneling of Kun Project and Thaukyegat Project

4.4.1 Introduction

In this study, focuses on geological assessment of two projects for making comparison which is located in the complex geology area and implemented by different parties. The projects are located in very weak geology area which engineering challenges of different tunneling methods in different geological conditions of rocks and solving the problems daily encountered during under construction. Detailed description and basic feature of the projects, and layout of the structures are already mentioned in above. Following table shows the comparison of general features of tunneling on Kun and Thaukyegat project.

Table 4.6: Comparison of Tunneling on Kun and Thaukyegat Project

Situation	KUN	Thaukyegat
1. Location		
(1) Sittaung Vally River Basin	Downstream most & West to Sittaung River	Middle Downstream& East to Sittaung River
2. Geological Condition		
(1) Lithology	Meta-sandstone, Mudstone (weak)	Phyllite, Schist, Meta-sandstone, (weak)
3. Structure		
(1) Diversion (W x H)	1.5 x 3.8 m (Conduit)	531 x 10 x 10.5 m (Tunnel)
(2) Headrace Tunnel (L x Diameter)	1755 x 5.5 m	538 x 8.5 m
4. Power Indices		
(1) Installed Capacity (MW)	60	120
5. Organization		
(1) Implementation by	Ministry of Electric Power (MOEP)	Gold Energy Co., Ltd (Local Company)

4.4.2 Geological Investigation and Assessment on Tunneling

Kun Project

According to geological assessment in preconstruction stage:

- Regional Geology: Early Mioscene Age, consisting of fine to medium grained, medium hard Sandstones interbedded with minor Shale and Siltstone.
- Regional metamorphism: Low to medium grade

- Waterway Tunnel:

  Lithology - Consists of Metasedimentaries

  (Metasandstone, Mudstone, Siltstone)

  Rock Structure- Striking N-S and N20°W with dipping (10°~ 4°)

  Rock Mass - Expected RMR ranges 25 to 70

Geology maps of waterway tunnel are prepared by on ground survey and core drill results as illustrated in Figure 4.23 (KEPCO, 2005).

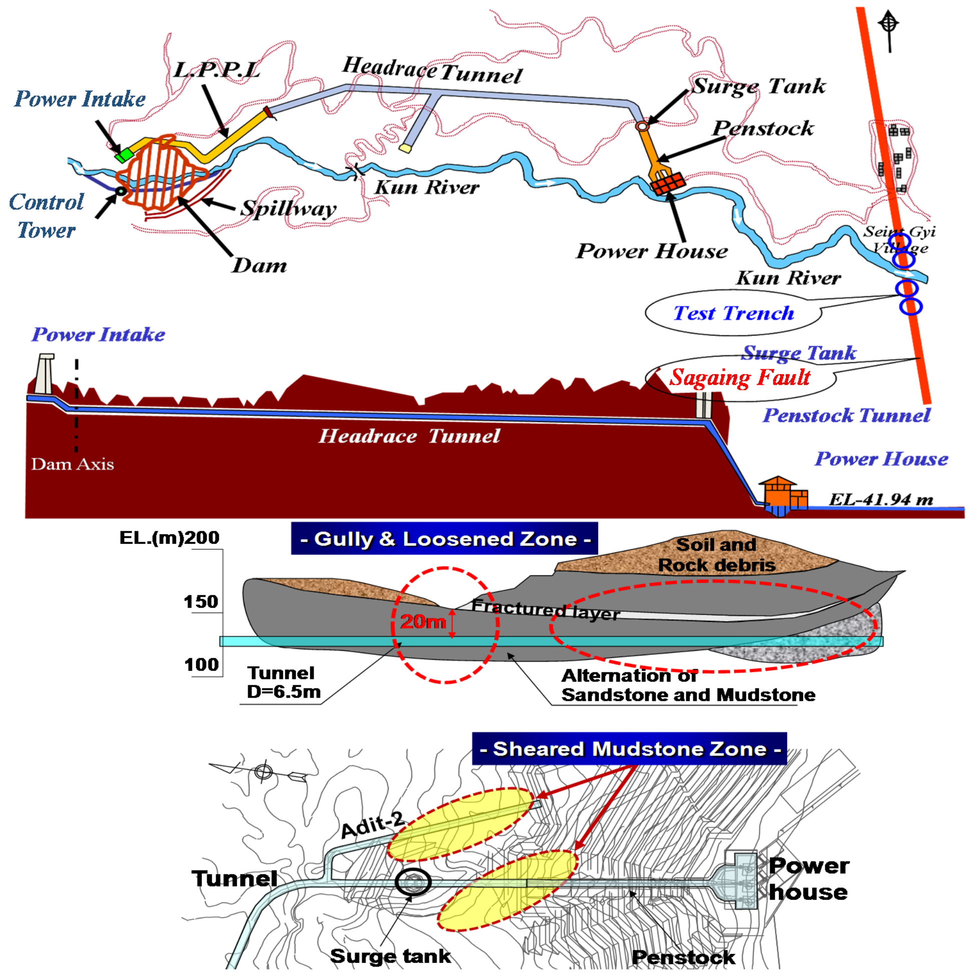


Figure 4.23: Geological Condition on Waterway Tunnel of Kun (Source: KEPCO, 2008)

Thaukyegat Project

According to geological assessment in preconstruction stage:

- Regional Geology: Paleozoic metamorphic rocks consisting of phyllite, quartzite andschist are distributed together with granites which sandwiches the metamorphic rocks.

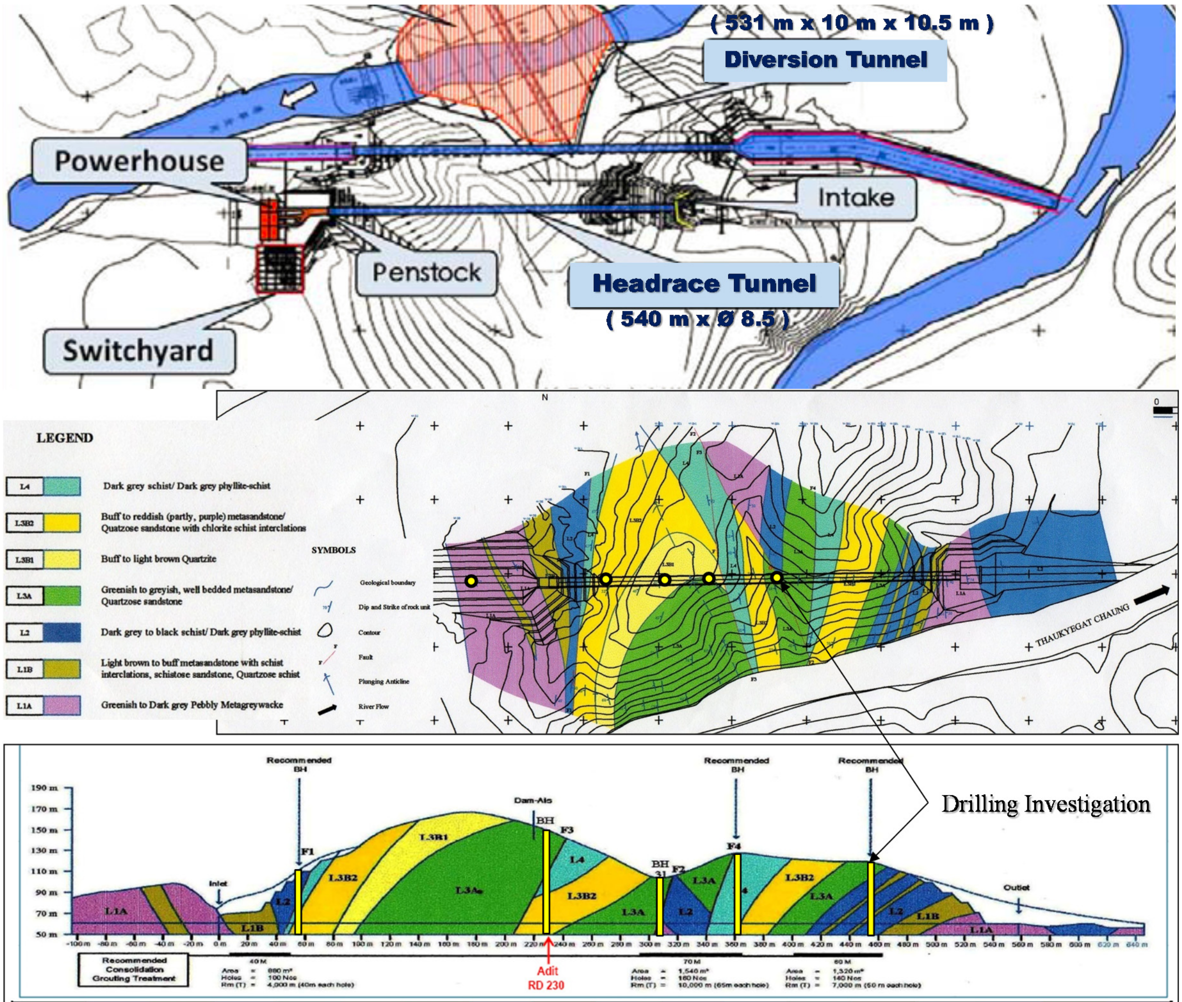
- Regional metamorphism: Low to medium grade
- Diversion Tunnel and Waterway Tunnel:

  Lithology - Consists of Metasedimentaries (Phyllite, Metasandstone, Quartzose sandstone, Schist, Pebbly Metagrey-wacke)

  Rock Structure - N-S to NNE-SSW with generally East ward dipping (45°~ 65°)

  Rock Mass - Expected Q value ranges 0.5 to 5 and RMR ranges 35 to 56

Geology maps of diversion tunnel and waterway tunnel are prepared by on ground survey and core drill results as illustrated in Figure 4.24 and 4.25 (Eyn Keey, 2012).



RD, m	RD 0 - 10	10 - 40	40 - 45	45 - 48	48 - 80	80 - 95	95 - 205	205-230	230 - 303	303 - 320	320 - 340	340 - 360	360 - 382	382 - 408	408 - 460	460 - 490	490 - 525
Length, m	10	30	5	3	32	15	110	25	73	17	20	20	22	26	52	30	35
Rock Symbol	L1A	L1B	L2	L4	L3B2	L3B1	L3A	L3B2	L3A	L1A	L2	L4	L3B2	L3A	L2	L1B	L1A
Weathering	HW	MW	MW-HW	HW	MW	SW-MW	SW	MW	SW	HW	MW-HW	HW	MW	SW	MW-HW	MW	HW
Remarks	Very weak	Very weak	Very weak	Very weak	Weak	Good	Fair	Weak	Fair	Very weak	Very weak	Very weak	Weak	Fair	Very weak	Very weak	Very weak

Note; HW = Highly weathering, MW = Medium Weathering, SW = Slightly weathering, MW- HW = Medium to Highly weathering, SW-MW = Slightly to medium weathering

Figure 4.24: Geological Condition on Diversion Tunnel of Thaukyegat

(Source: Eyn Keey, 2012)

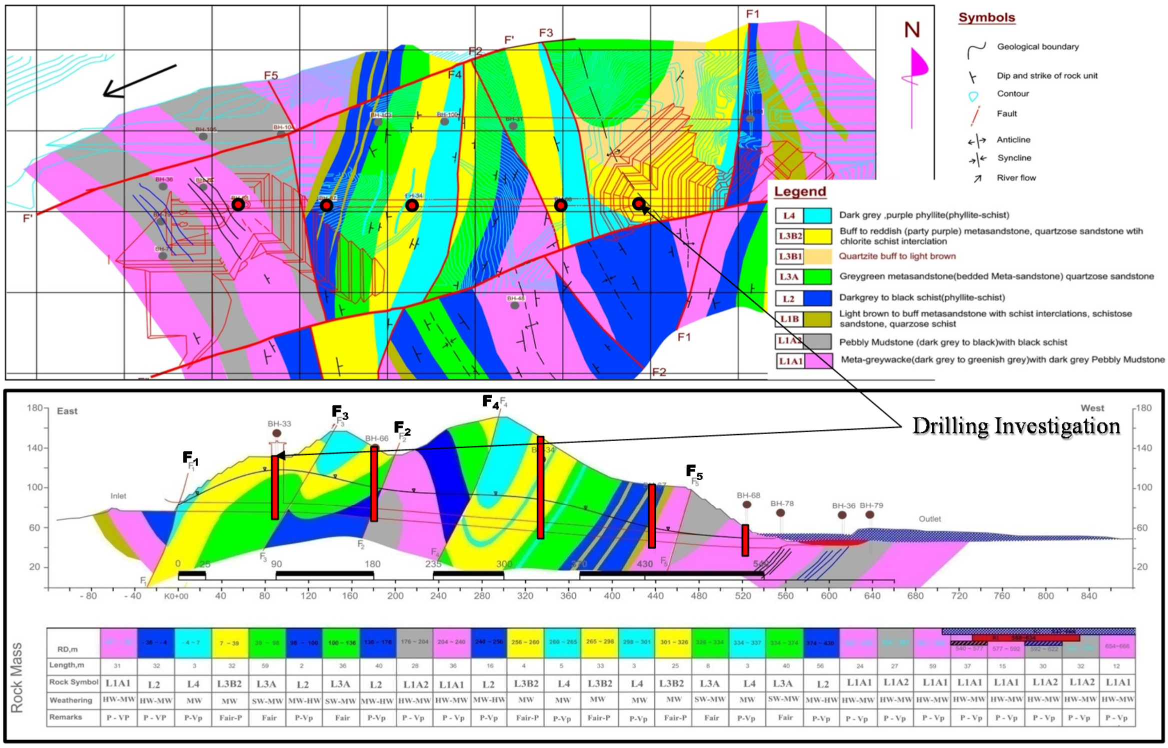


Figure 4.25: Geological Condition on Waterway Tunnel of Thaukyegat

(Source: Eyn Keey, 2012)

To provide a quantitative estimate of rock mass quality from drill logs and tunnel excavated face, multi parameter rock mass classification schemes have been used in selecting the tunnel excavation methods, rock supporting methods and blasting patterns in the tunneling works on both projects. Kun tunneling excavation practice is adopted RQD and RMR system, and Thaukyegat is adopted RSR, RMR System and Q Index. The procedure for measurement and calculation of RQD, RMR and Q Index are already presented in above.

4.4.3 Evaluation on Complex Geology and Difficulties of Kun Tunneling

For Kun waterway tunnel, the significant problems and each excavated faces were recorded which includes both of data collection and assessment that revealed from construction stage and quality assessment of various performances of construction stages for in-situ engineering geology and reasonable ground treatment in the course of the tunneling. RMR system and RQD methods were used in selecting the tunnel excavation methods, rock supporting methods and blasting pattern. For the procedure of tunnel excavation, Conventional Method and New Austrian Tunneling Method (NATM) are widely used during tunneling. Considering the workability and quality of tunnel work under normal geological condition, NATM is generally superior to the Conventional Method. However, in the tunneling of Kun

project, the Conventional Method is prior to NATM due to the problem complex geological situations. Figure 4.26 shows the engineering geological record sheet and design supporting patterns are demonstrated in Figure 4.27 of Kun project (Maw Thar Htwe, 2009). Table 4.7 presents the RMR classification on Kun tunneling prepared by localized geological situations referring (Bieniawski, 1975) geomechanics classification (KEPCO, 2005).

Table 4.7: RMR Classification on Kun Tunneling

A. RMR Classification (after Bieniawski, 1975a)

1	Strength of intact rock	Uniaxial Compressive Strength	>200MPa	100-200MPa	50-100MPa	25-50MPa	10-25MPa	3-10MPa	1-3MPa
	Rating		15	12	7	4	2	1	0
2	RQD		90-100%	75-90%	50-75%	25-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>3m	1-3m	0.3-1m	50-300mm	<50mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous Not separation Hard joint wall rock	Slightly rough surfaces Separation<1mm Hard joint wall rock	Slightly rough surfaces Separation<1mm Soft joint wall rock	Slickensided surfaces or Gouge<5mm thick or Joints open 1-5mm Continuous joints	Soft gouge>5mm thick or Joint open>5mm Continuous joints		
	Rating		25	20	12	6	0		
5	Groundwater	General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water pressure		
	Rating		10		7	4	0		

B. Adjustment for Joint Orientations

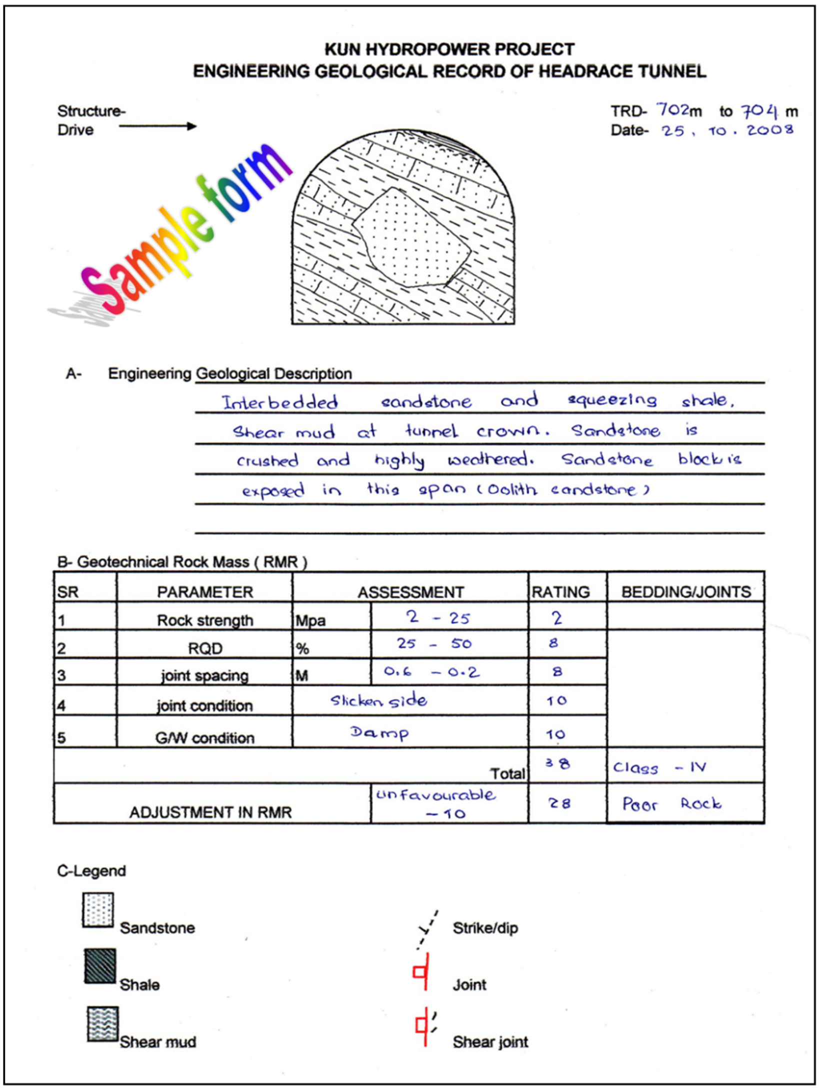
Strike and dip		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnel	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. Rock Mass Classes and their Ratings

Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair tock	Poor rock	Very poor rock
Rating	100-90	90-70	70-50	50-25	<25

D. Meaning of Rock Mass Classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 5m span	6 months for 4m span	1 week for 3m span	5 hours for 1.5m span	10 minutes
Cohesion of the rock mass	> 300kPa	200-300kPa	150-200kPa	100-150kPa	< 100kPa
Friction angle of the rock mass	> 45°	40-45°	35-40°	30-35°	< 30°



KUN HYDROPOWER PROJECT
ENGINEERING GEOLOGICAL RECORD OF HEADRACE TUNNEL

Structure-
Drive

TRD- 702m to 704 m
Date- 25 . 10 . 2008

Sample form

A- Engineering Geological Description

Interbedded sandstone and squeezing shale, Shear mud at tunnel crown. Sandstone is crushed and highly weathered. Sandstone block is exposed in this span (Oolith sandstone)

B- Geotechnical Rock Mass (RMR)

SR	PARAMETER	ASSESSMENT		RATING	BEDDING/JOINTS
1	Rock strength	Mpa	2 - 25	2	
2	RQD	%	25 - 50	8	
3	joint spacing	M	0.6 - 0.2	8	
4	joint condition	Slicken side		10	
5	G/W condition	Damp		10	
			Total	38	Class - IV
	ADJUSTMENT IN RMR		unfavourable -10	28	Poor Rock

C-Legend

Sandstone
Shale
Shear mud
Strike/dip
Joint
Shear joint

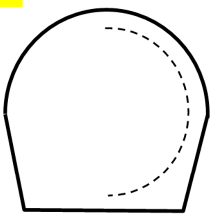
Figure 4.26: Engineering Geological Record Sheet of Kun Project

(Source: Maw Thar Htwe, 2009)

Very Good Rock

RMR Rating 90 ~ 100
RQD% 90 ~ 100

No need supporting
No need shotcreting

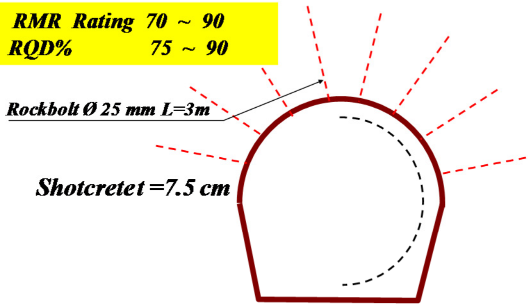


Rock Determination

Rock Condition	RMR (Rating)
Very Good	**I (90~100)**
Good	**II (70~90)**
Fair	**III (50~70)**
Poor	**IV (25~50)**
Very Poor	**V (<25)**

One Advance Length (m)	*Shotcrete Thickness (cm)*	*Rockbolt*			*Wiremesh*	*Steel Arch*	*Method of Excavation*
		Length (m)	*Pitch* *Circumference (m)*	*Pitch* *Longitudinal (m)*			
1.5 ~ 3	-	-	-	-	-	-	*Full face excavation by blasting*

Good Rock

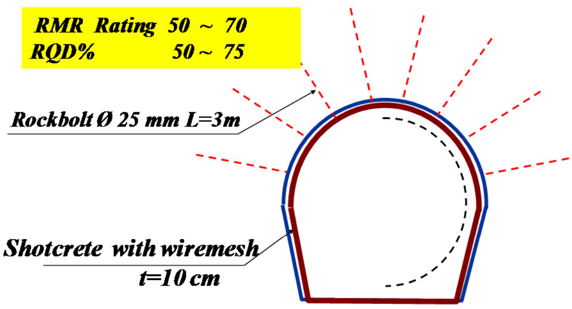


Rock Determination

Rock Condition	RMR (Rating)
Very Good	I (90~100)
Good	II (70~90)
Fair	III (50~70)
Poor	IV (25~50)
Very Poor	V (<25)

One Advance Length (m)	Shotcrete Thickness (cm)	Rockbolt			Wiremesh	Steel Arch	Method of Excavation
			Pitch				
		Length (m)	Circumference (m)	Longitudinal (m)			
1.5	7.5	3	1.2	1.5	-	-	Full face excavation by blasting

Fair Rock

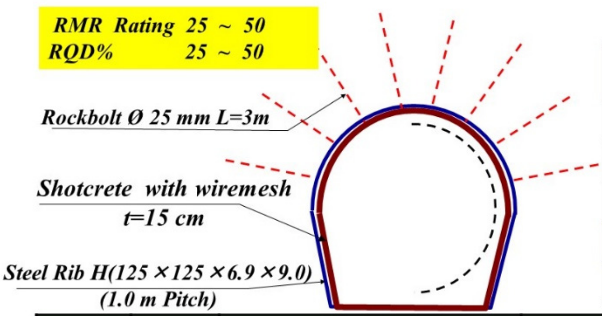


Rock Determination

Rock Condition	RMR (Rating)
Very Good	I (90~100)
Good	II (70~90)
Fair	III (50~70)
Poor	IV (25~50)
Very Poor	V (<25)

One Advance Length (m)	Shotcrete Thickness (cm)	Rockbolt			Wiremesh	Steel Arch	Method of Excavation
			Pitch				
		Length (m)	Circumference (m)	Longitudinal (m)			
1.2	10	3	1.2	1.2	150 × 150 × Ø5 (upper portion)	-	Full face excavation by blasting

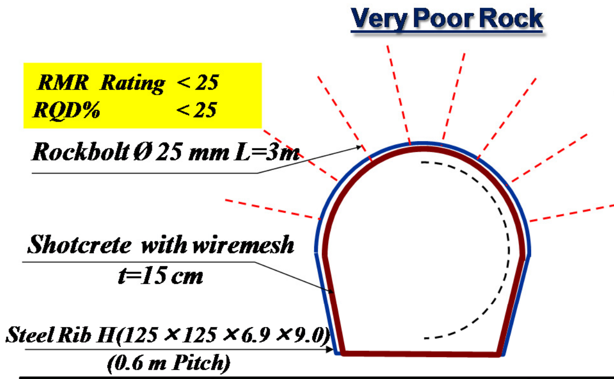
Poor Rock



Rock Determination

Rock Condition	RMR (Rating)
Very Good	I (90~100)
Good	II (70~90)
Fair	III (50~70)
Poor	IV (25~50)
Very Poor	V (<25)

One Advance Length (m)	Shotcrete Thickness (cm)	Rockbolt			Wiremesh	Steel Arch	Method of Excavation
			Pitch				
		Length (m)	Circumference (m)	Longitudinal (m)			
1	15	3	1.2	1	150 × 150 × Ø5	H-125 × 125 × 6.5 × 9.0 1.0 m pitch	excavated by blasting or manual



Rock Determination

Rock Condition	RMR (Rating)
Very Good	I (90~100)
Good	II (70~90)
Fair	III (50~70)
Poor	IV (25~50)
Very Poor	V (<25)

One Advance Length (m)	Shotcrete Thickness (cm)	Rockbolt	Rockbolt Pitch		Wiremesh	Steel Arch	Method of Excavation
		Length (m)	Circumference (m)	Longitudinal (m)			
0.6	15	3	1.2	1	150 × 150 × Ø5	H-125 × 125 × 6.5 × 9.0 0.6 m pitch	Ring cut by manual

Figure 4.27: Determination of Rock Mass and Tunnel Supporting Systems

(Source: Maw Thar Htwe, 2009)

Half of the total length of waterway can be explored by conventional method of tunneling and remaining half was excavated by different auxiliary methods by lowering down the momentum of the working speed comparatively. The most difficult places along the tunnel alignment are presented in following figures (KEPCO, 2005).

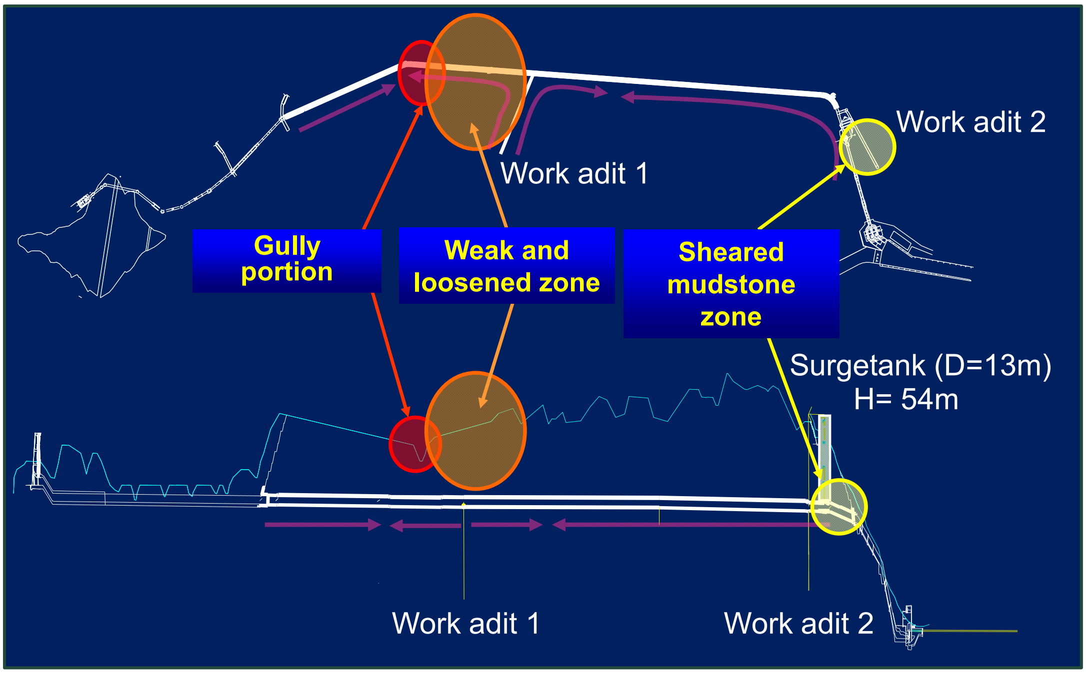


Figure 4.28: Geological Condition on Waterway Tunnel of Kun Project

(Source: KEPCO, 2008)

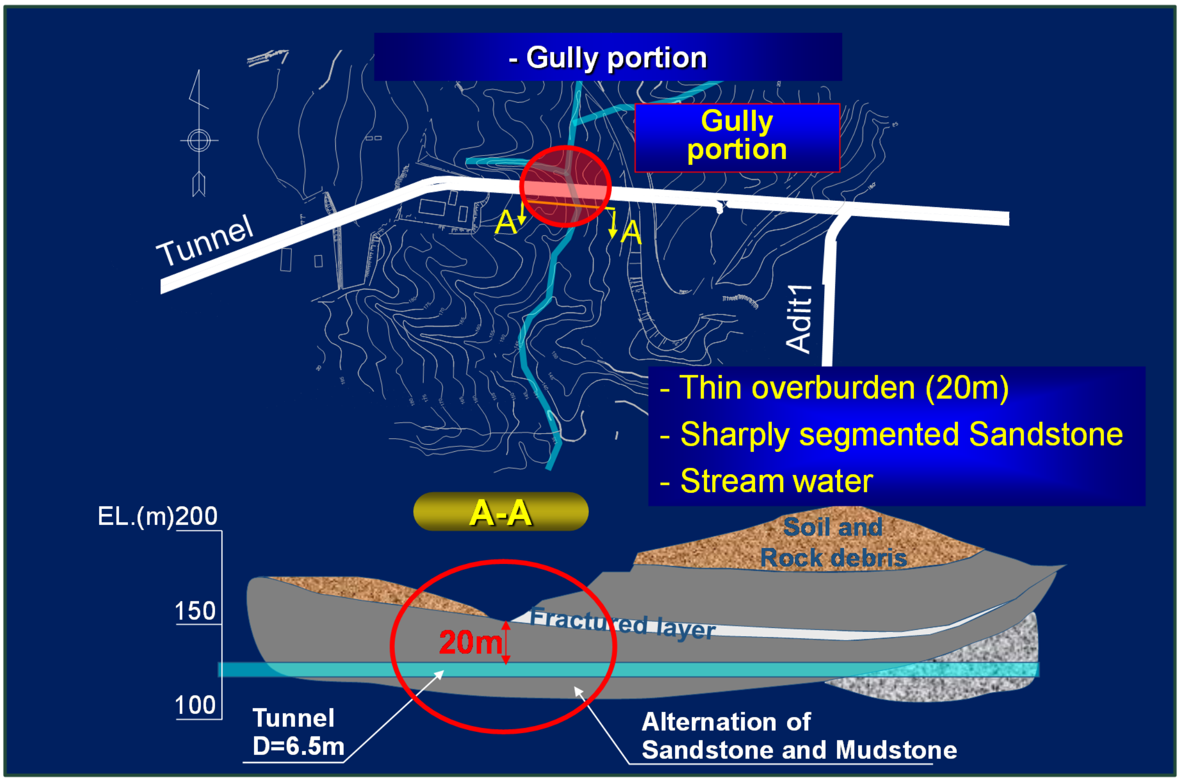


Figure 4.29: Geological Condition of Gully Portion of Kun Project

(Source: KEPCO, 2008)

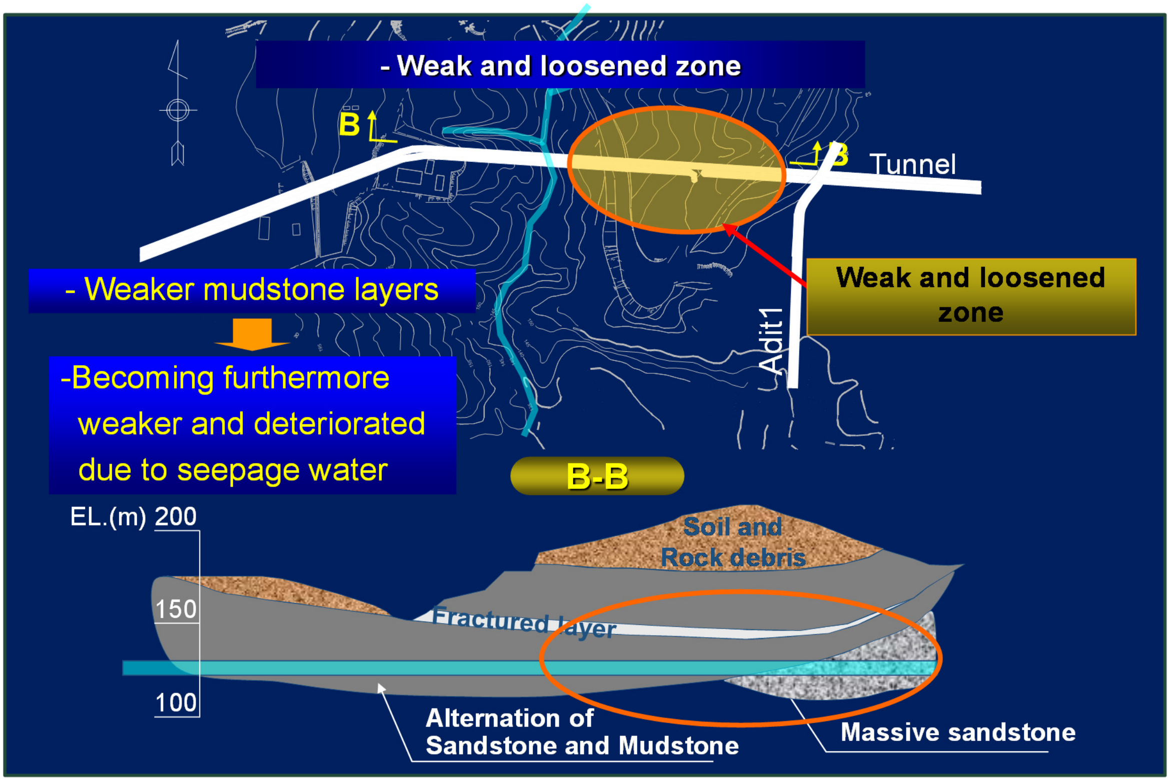


Figure 4.30: Geological Condition of Weak and Loosened Zone of Kun Project

(Source: KEPCO, 2008)

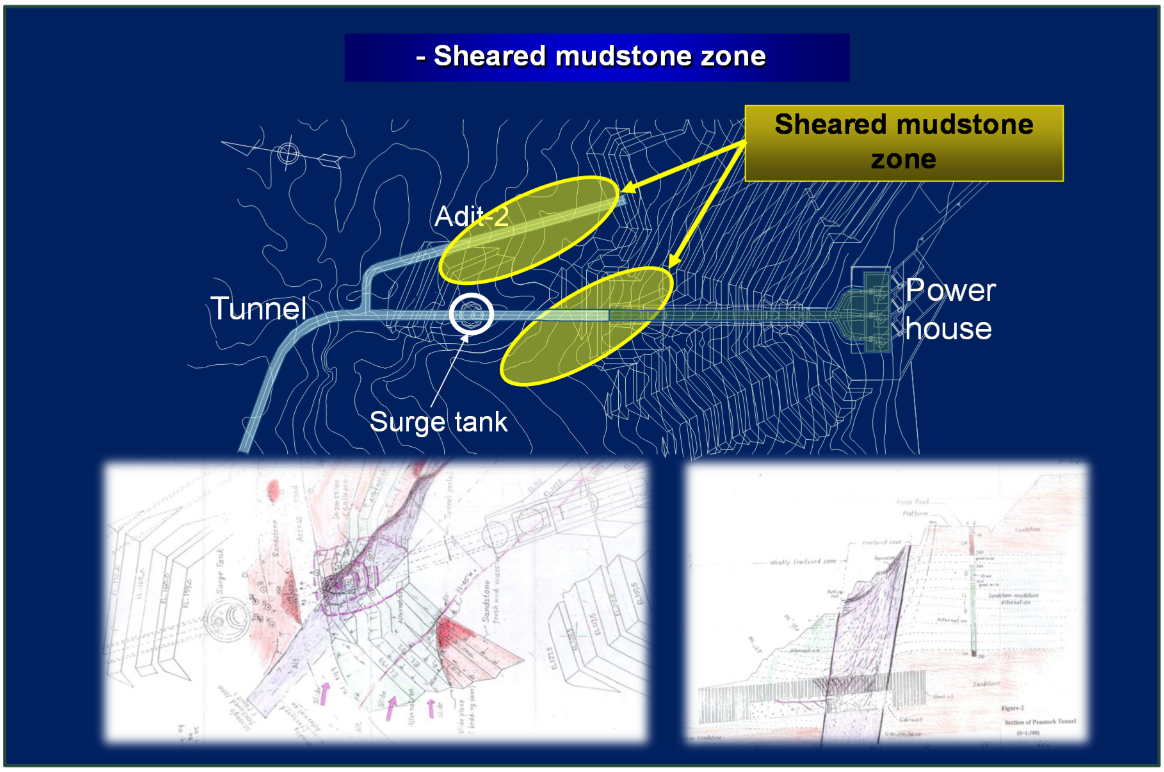


Figure 4.31: Geological Condition of Sheared Mudstone Zone of Kun Project

(Source: KEPCO, 2008)

According to rock mass record along the waterway tunnel alignment, half of the tunnel alignment area is poor to fair and just very small portion exist good geology condition as demonstrated in Figure 4.32 and 4.33.

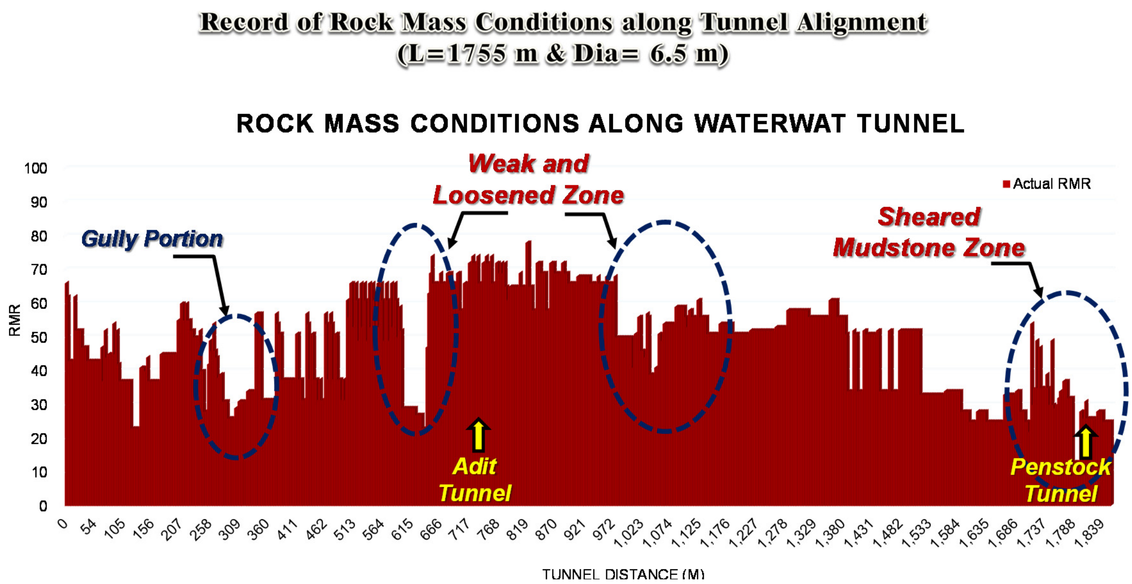


Figure 4.32: Rock Mass Condition along Waterway Tunnel of Kun Project

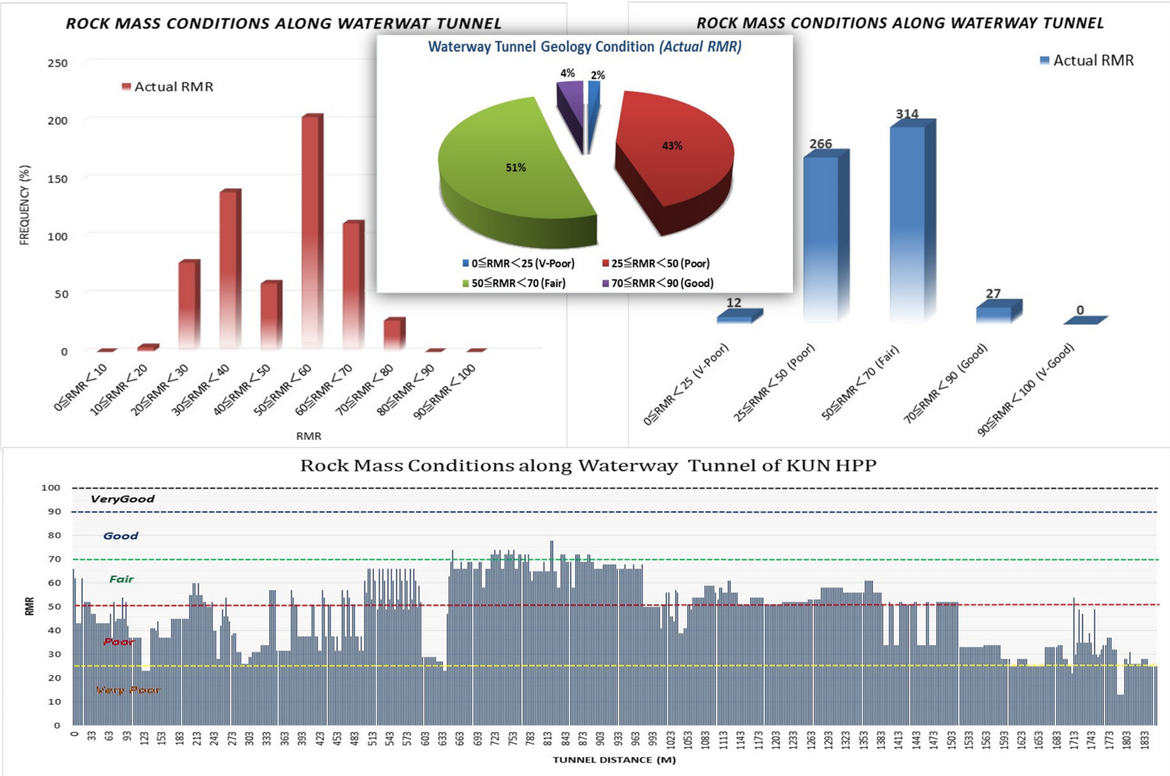


Figure 4.33: Quality of Rock Mass along Waterway Tunnel of Kun Project

4.4.4 Evaluation on Complex Geology and Difficulties of Thaukyegat Tunneling

For Thaukyegat project, both of diversion tunnel and waterway tunnel are existed hidden instability modes which is highly to moderate weathered rocks along the course of tunnel exploration. Every temporary supporting system is necessary to cover the “stand up” time during excavation on both tunnels. RSR, RMR System and Q Index methods were used in selecting the tunnel excavation methods, rock supporting methods and blasting pattern. According to rock mass record along the diversion tunnel alignment, most of the tunnel area is very poor to poor and just some portion exist fair geology condition, and some failure cases are demonstrated in Figure 4.34 and 4.35.

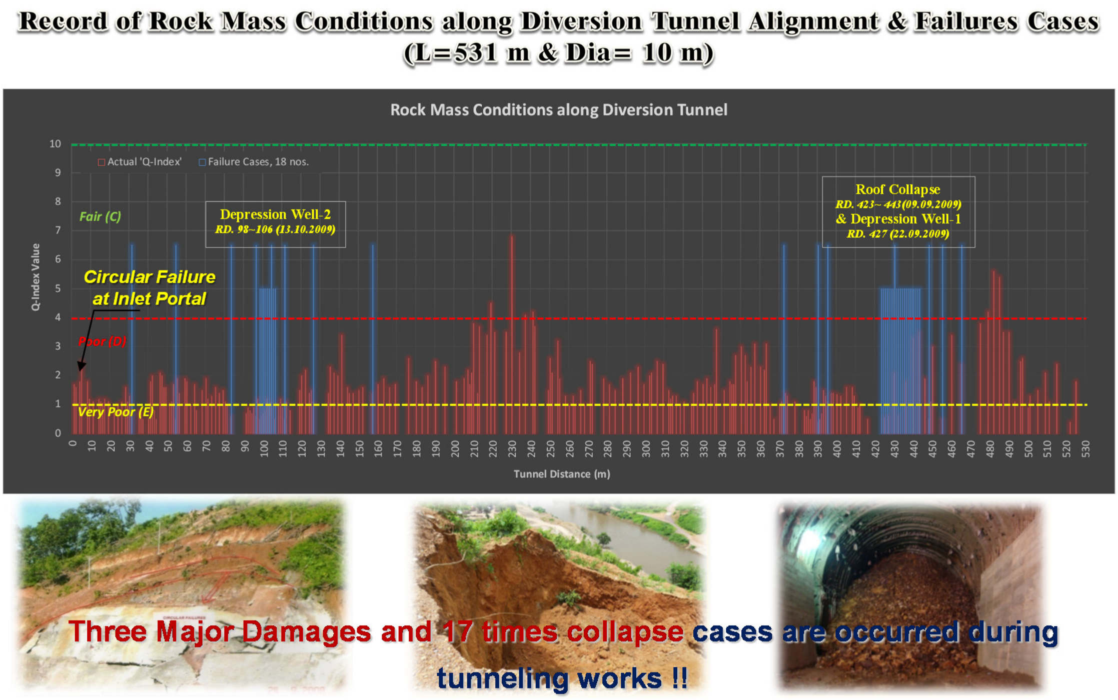


Figure 4.34: Rock Mass Condition along Diversion Tunnel of Thaukyegat Project

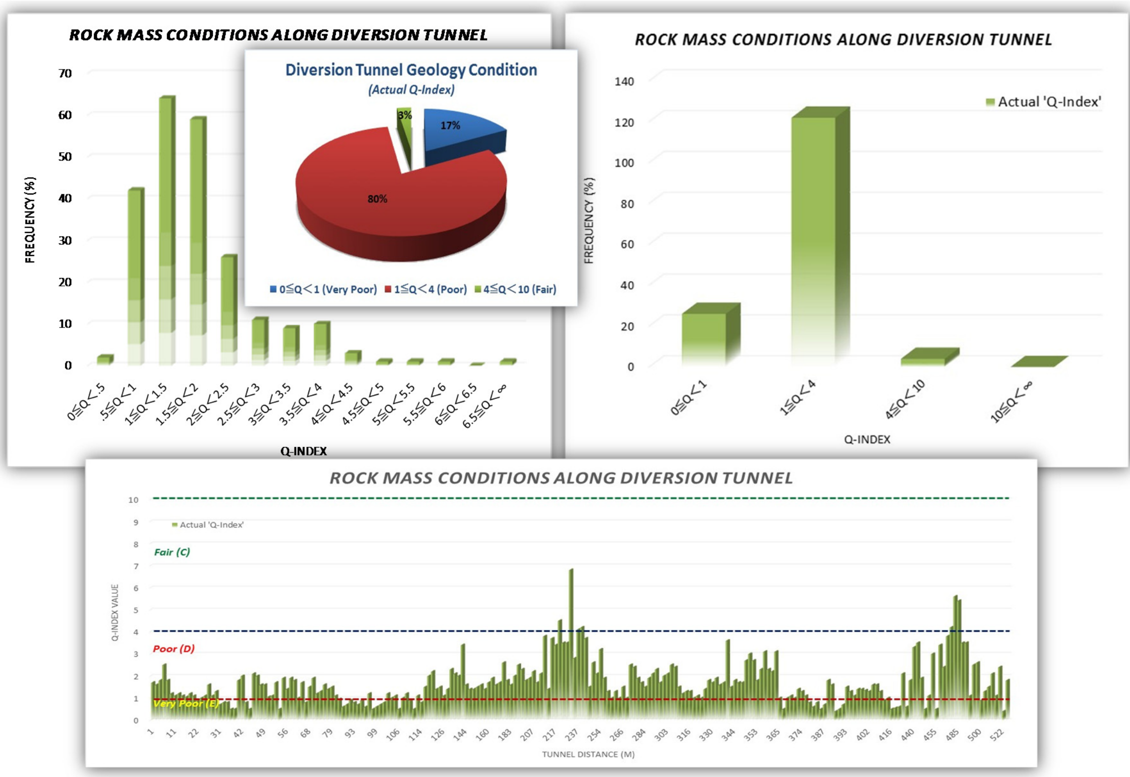


Figure 4.35: Quality of Rock Mass along Diversion Tunnel of Thaukyegat Project

Before waterway tunnel exploration, pre-geological assessment on tunnel alignment and setting on design Q index for whole tunnel by referring diversion tunnel rock mass record and practices. During the waterway tunnel excavation, there is no serious collapse cases

except small scaled twelve times failure cases along the tunnel where is owing to condition of higher weathering degree of rock masses under the condition of ground water. However, by mean of well preparation and better geological assessment on tunneling works, there is no serious failures cases are occur and construction progress is proceeding well. According to rock mass record along the waterway tunnel alignment, one-third of the tunnel area is very poor and two-third is poor, and just small portion exist fair geology condition as demonstrated in Figure 4.36 and 4.37.

Record of Rock Mass Conditions along Waterway Tunnel Alignment & Failures Cases (L=540 m & Dia= 8.5 m)

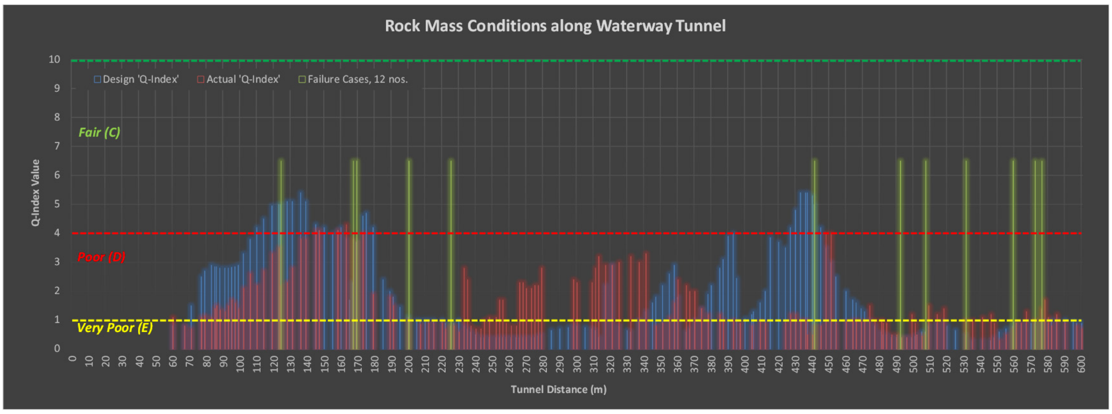


Figure 4.36: Rock Mass Condition along Waterway Tunnel of Thaukyegat Project

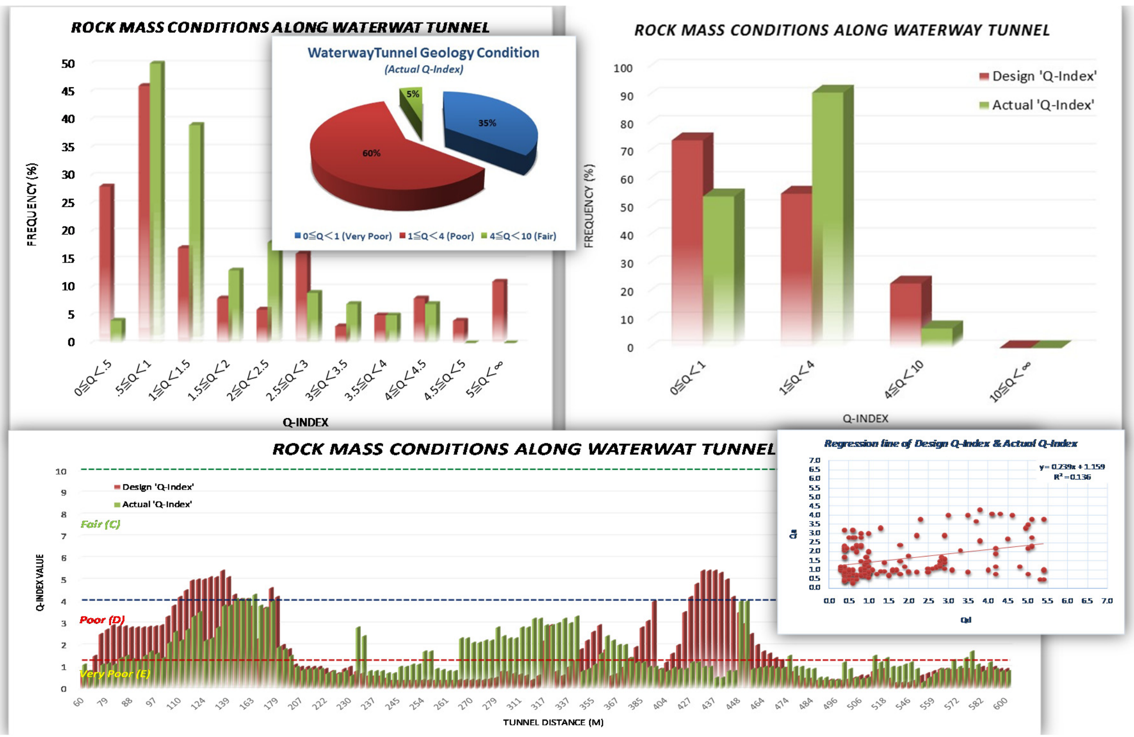


Figure 4.37: Quality of Rock Mass along Waterway Tunnel of Thaukyegat Project

Comparison of diversion tunnel and waterway tunnel, general impressions of rock quality are almost very poor to poor condition on both tunnel alignment and relatively similar on each other because of located in the same mountain as illustrated in Figure 4.38.This situation is favorable for later exploration of waterway tunnel with less risk on tunnel collapse cases. According to regression line of both tunnels, it can be easily understood about the unforeseeable of geological conditions even though existed in the same mountain.

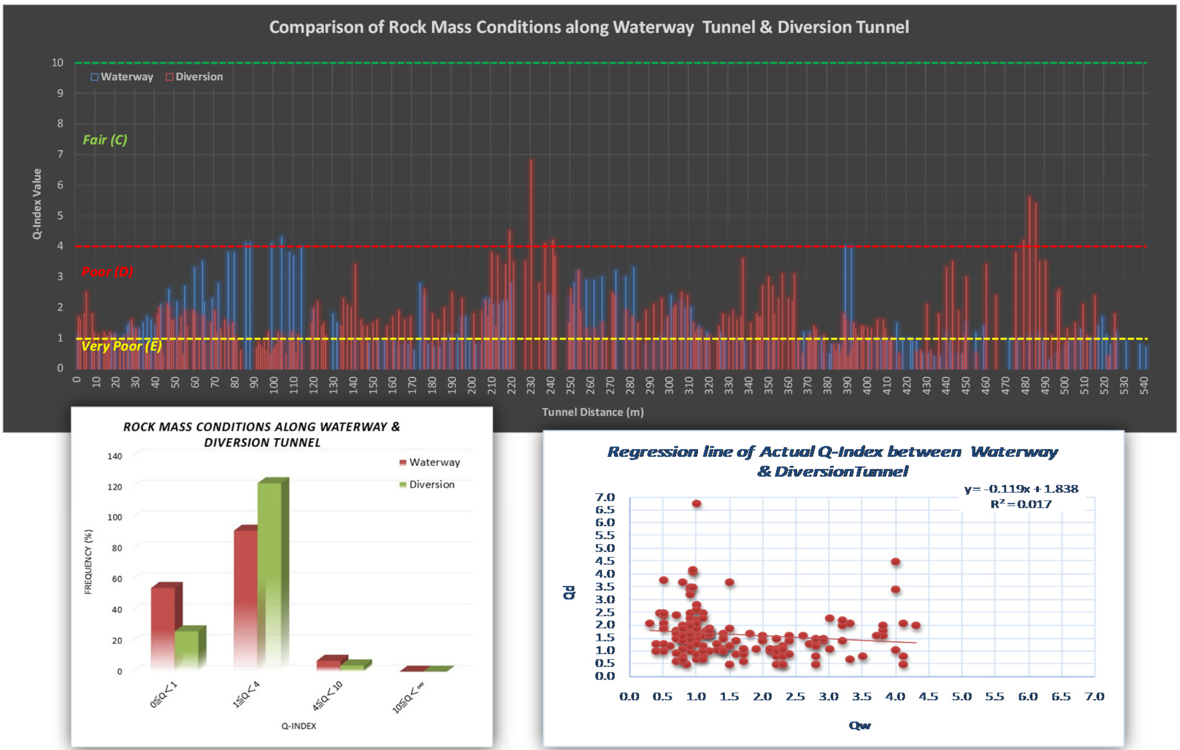


Figure 4.38: Comparison on Waterway and Diversion Tunnel of Thaukyegat Project

In the course of tunnel excavation on Thaukyegat project where the existent of difficulties and complicated geological conditions, the problems and issues faced from time to time and tackling measures are emphasized and overcome by introducing auxiliary methods. Furthermore, it is applied more complicated sequence and procedure for tunneling.

4.4.5 Tunnel Failure Mechanisms of Kun Project and Thaukyegat Project

Kun Project

The serious two major collapses were occurred at penstock tunnel where sheared mud zone and the other five serious failure cases are also faced along the waterway tunnel as demonstrated in Figure 4.39. The rock mass conditions of serious and complicated geological situations on penstock tunnel where shear mudstone zone is existed are illustrated

in Figure 4.40. During the waterway tunnel excavation of Kun project, the complicated problems and issues were faced and couples of working days were lost but construction achievement is different concerning with budget and time schedule of the projects.

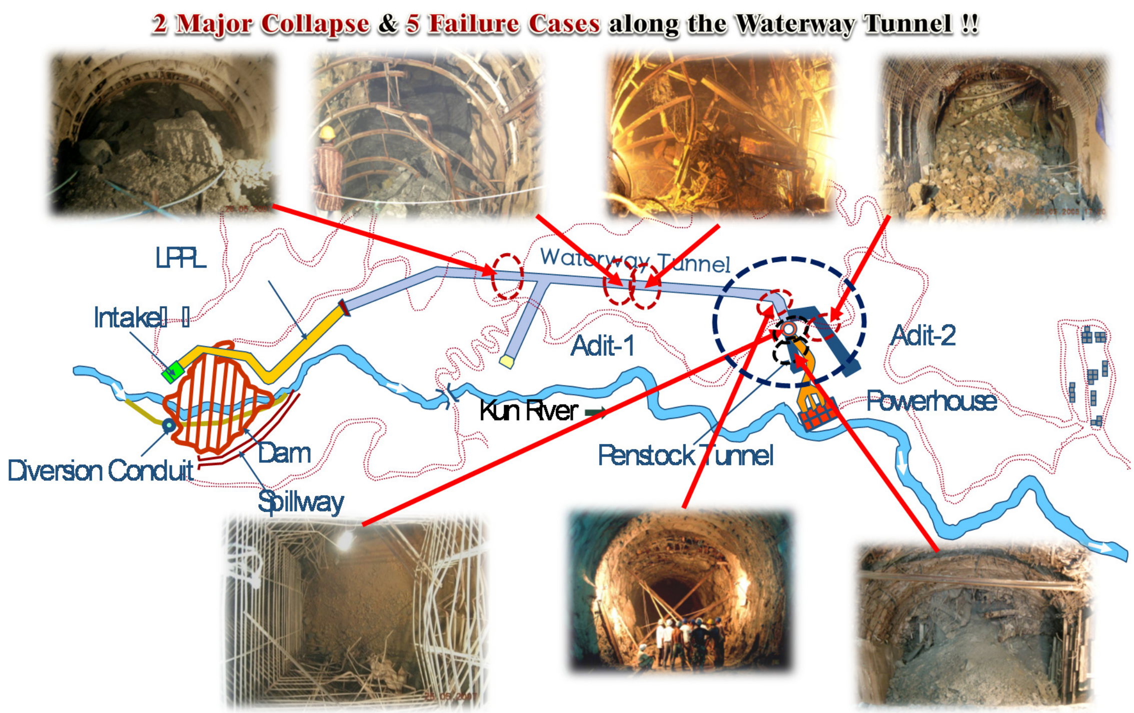


Figure 4.39: Waterway Tunnel Failure Cases of Kun Project

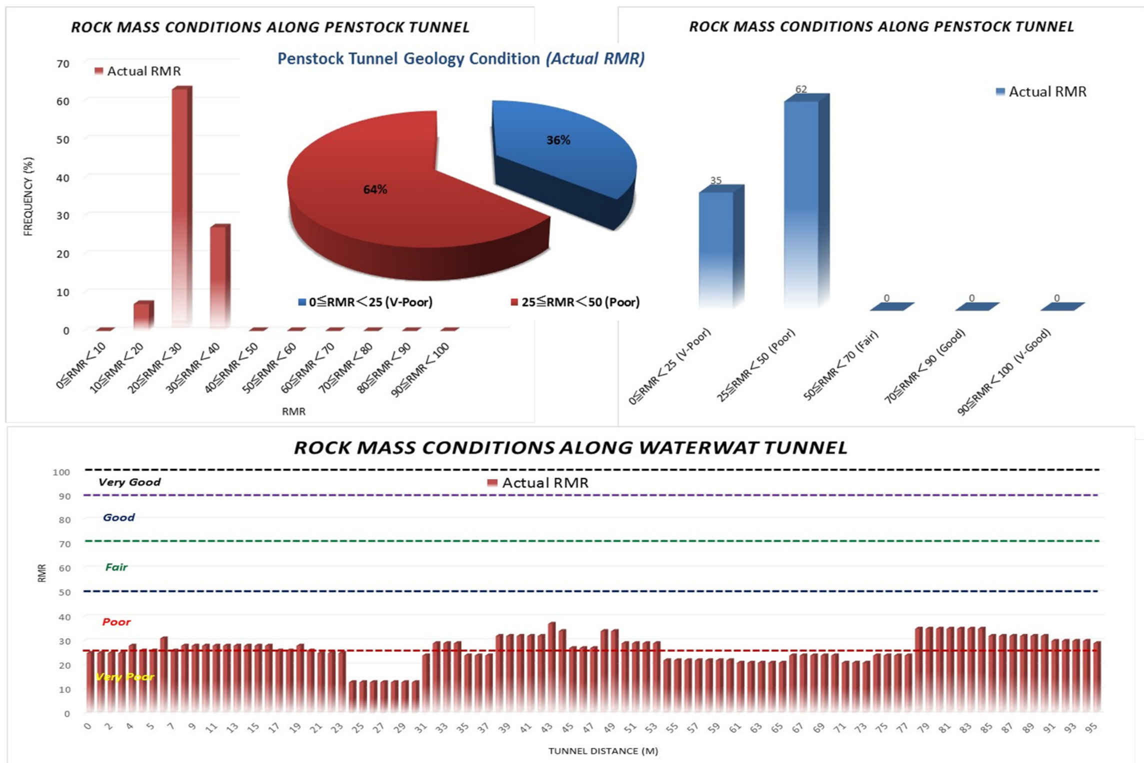


Figure 4.40: Quality Rock Mass along New Penstock Tunnel of Kun Project

At weak and loosened zone between the upstream of work adit (1) and junction, the face failure was occurred because of fractured layer which was becoming weaker and deteriorated due to seepage water. The lithology of loosened face condition and failure mechanisms are illustrated in Figure 4.41.

At fractured zone downstream of work adit (1), the roof wedge failure was occurred because of fractured layer with unfavorable joint set. The rock class on the downstream of the junction of work adit (1) were fair rocks which the rock mass rating was about 40 to 50 RMR. However, vertical joints were appeared and parallel with tunnel alignment, and collapsed. The lithology of fractured face condition and failure mechanisms are illustrated in Figure 4.42.

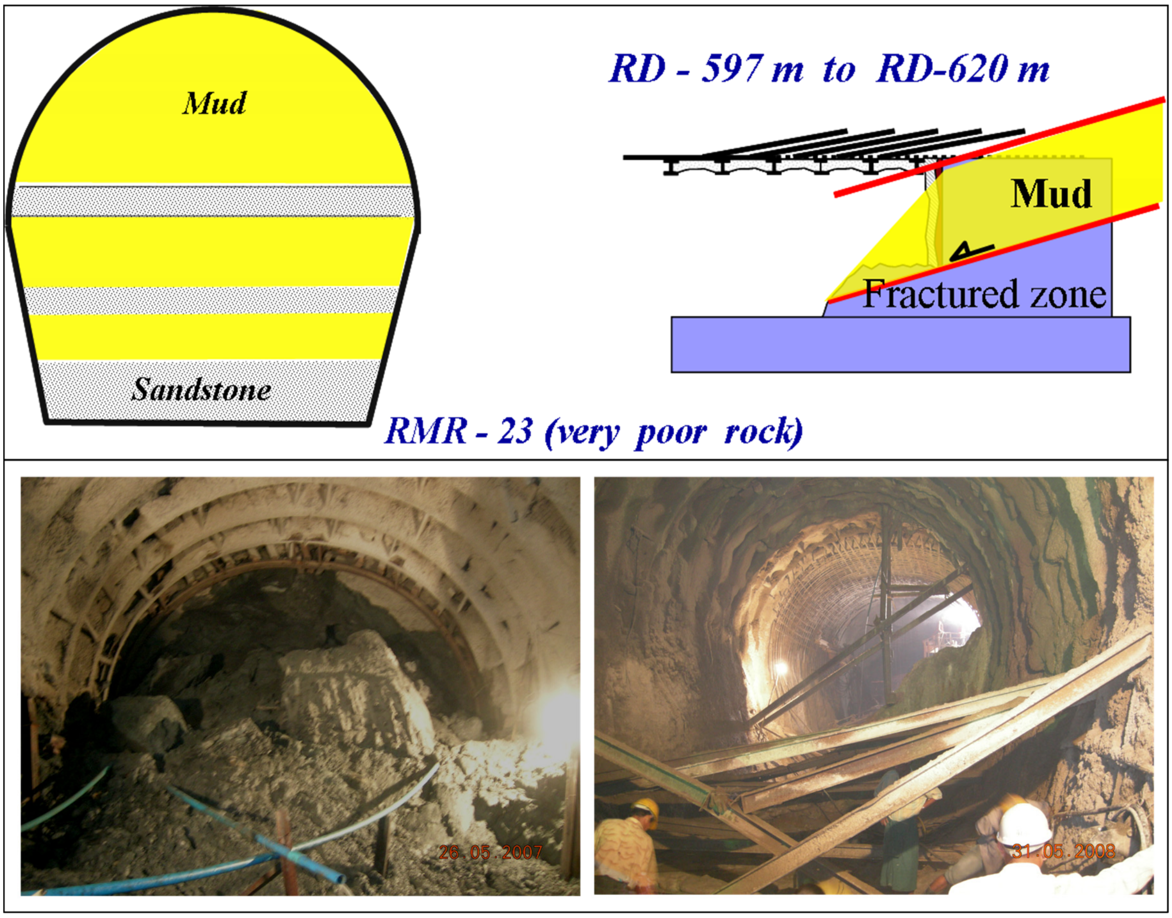


Figure 4.41: Tunnel Face Failure at Weak and Loosened Zone

(Source: Maw Thar Htwe, 2009)

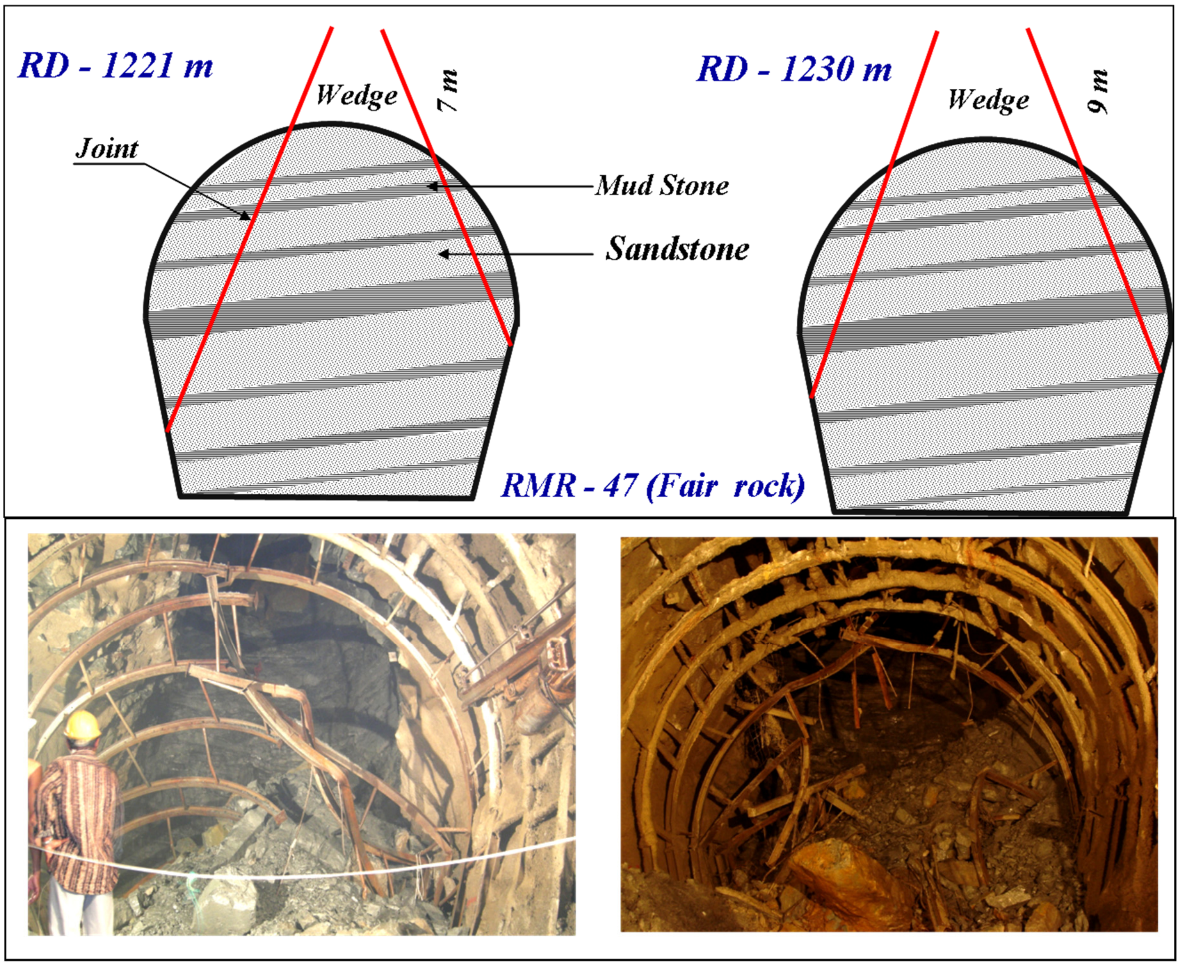


Figure 4.42: Tunnel Crown Failure at Fractured Zone

(Source: Maw Thar Htwe, 2009)

At sheared mudstone zone between surge tank and penstock tunnel, the face failure was occurred because of fractured layer which composed of massive sandstone and sandstone-mudstone alternation. When the penstock tunnel collapsed the surface of mountain cutting slopes were depressed and deformed, just above the portion of tunnel collapse and the slope on mountain side was pushed out. Even though consolidation grouting was implement along the mountain slope on this alignment, it could not be treated and improved the shear mud layer. Therefore, the alternative penstock tunnel alignment was chosen left side of the existing alignment of collapsed area. The lithology of fractured face condition and failure mechanisms are illustrated in Figure 4.43.

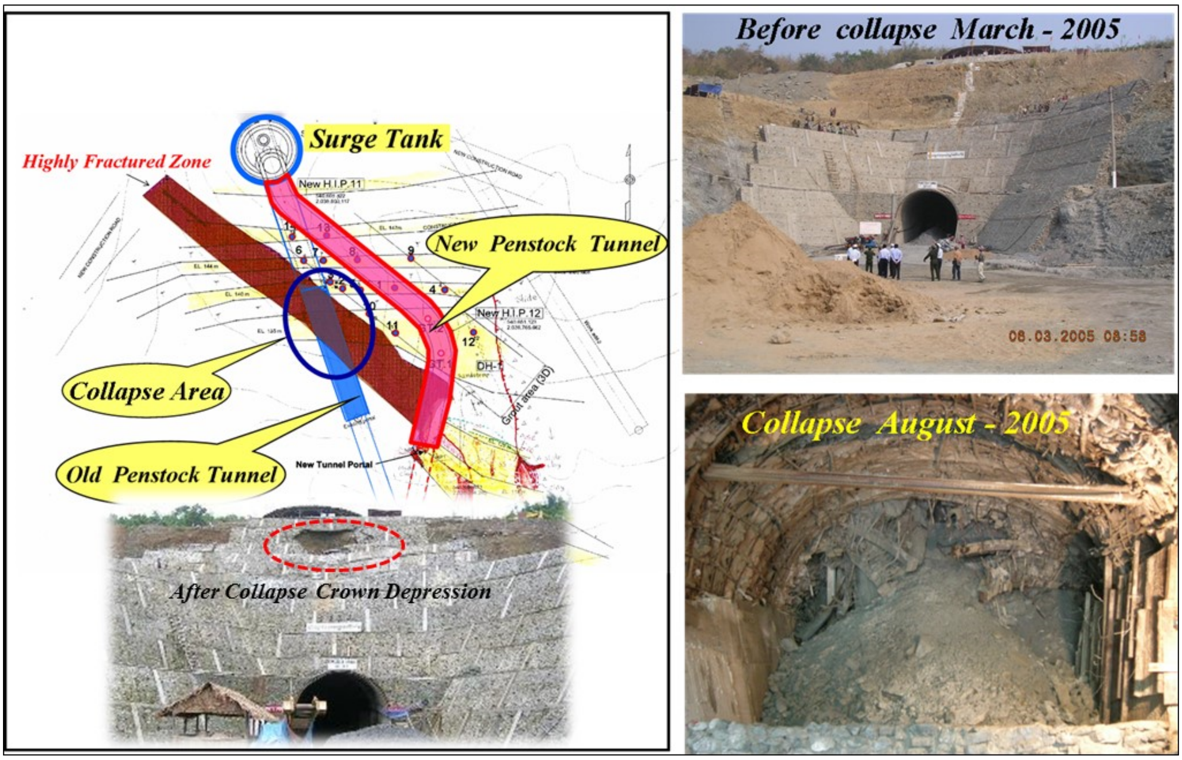


Figure 4.43: Tunnel Face Failure at Sheared Mudstone Zone

(Source: Maw Thar Htwe, 2009)

Thaukyegat Project

During the diversion tunnel excavation, the serious three major collapses were occurred around inlet and outlet where is owing to condition of higher weathering degree of rock masses under the condition of ground water, and the other twenty times failure cases are also faced during exploration. Along the diversion tunnel, the most frequent failure cases are plain failure and roof wedge by means of fault line across the alignment and grown water high discharge, and the most difficult area is outlet area which was collapsed by face failure of depression well. Table 4.8 presents the failure cases along the diversion tunnel on Thaukyegat project (Eyn Keey, 2012). In the waterway tunnel also 12 times of tunnel failure cases are faced such as plain failure and roof wedge but mostly are minor failure and not much disturbed the tunnel excavation progress of the project.

Table 4.8: Record of Failure Cases along the Diversion Tunnel of Thaukyegat

Sr. No.	Location	Date	Time	RD (m)	Faliure Type	Estimate Volume (m^3)	Remark
1	Construction Adit			3 - 5	Collapse (right side of top)		Weak zone
2	Construction Adit	01 Dec.2008		63-78	Roof failure	439.74	Fault line across the alignment and GW high discharge
3	Inlet Slope	24 Sep.2008		EL-110 ~ EL-130	Circular failure (slopes)		Inlet portal slopes
4	Inlet	25 May.2009		31.5 - 32.0	Shear plane failure		
5	Inlet	22 July.2009		54.5	Plane failure		
6	Inlet	09 Sep.2009		84-89	Fault plane Failure		Break through point of Inlet and Adit inlet
	Break Through Point (Inlet & Adit-Inlet)			84			
7	Adit Inlet	13 Oct.2009	1:25 pm	98 - 106			Mixed Type of Depression Well
8	Adit Inlet	20 May.2009	7:30 am	157.3 - 159.8	Roof Wedge		(3.5 x 3.5 x 3.5)m^3 stone blk collapsed
9	Adit Inlet	29 Jun.2009		128.5 - 134.5	Plane failure		
10	Adit Inlet	16 Jul.2009		112.5	Roof Wedge		Large 3 blks of stone collapsed
11	Adit Inlet	05 Aug.2009		104	Plane failure		
12	Adit Inlet	24 Aug.2009		97.5	Plane failure		
	Junction Point of Adit & Diversion Tunnel			235			
13	Adit Outlet	22 Jul.2009		373.2	Roof Wedge		
	Break Through Point (Adit-Outlet & Outlet)			378.5			
14	Outlet	19 Dec.2008	11:00 pm to 03:00 am	464.8 - 477.3	Plane failure		
15	Outlet	7 Jan.2009	3:30 pm	453.3 - 457	Plane failure	167.5	
16	Outlet	27 Feb.2009		429.7 - 432.7	Plane failure		
17	Outlet	20 May.2009		391.8 - 394.8	Roof Wedge		
18	Outlet	14 Aug.2009		395	Plane failure		
19	Outlet	09 Sep.2009		423 - 442	Roof Collapse		
20	Outlet	22 Sep.2009	11:00 a.m	427	Depression Well /Sink		
21	Outlet	12 Feb.2009	10:30 pm	448-437	Roof Wedge		

Figure 4.44: Tunnel Face Failure by Depression Well (Source: Eyn Keey, 2012)

4.4.6 Comparison of Tunneling Progress on Kun Project and Thaukyegat Project

By making comparison of tunneling on Kun and Thaukyegat project, geological conditions on both projects are similar and highly risk on tunneling. For both projects, tunnel exploration cannot be speedily preceded under the situation of very poor geological conditions and unforeseen condition of the underground works. At outlet region of waterway tunnel of Kun project, special take care for the sheared mudstone zone and monthly progress is about 26 m per month. And then, at inlet and outlet region of diversion tunnel of Thaukyegat project, specially take care for highly weathered rock with ground water and monthly progress is about 18 m per month but other area can speed up to 71 m per month. Anyhow, waterway tunnel of Thaukyegat project is more progress than others about 108 m per month even though the same poor geological conditions as illustrated in Figure 4.45. So the systematic geological observation, procurement of tunnel tools and equipment are essential for excavation and supporting system on tunneling of weak geological area.

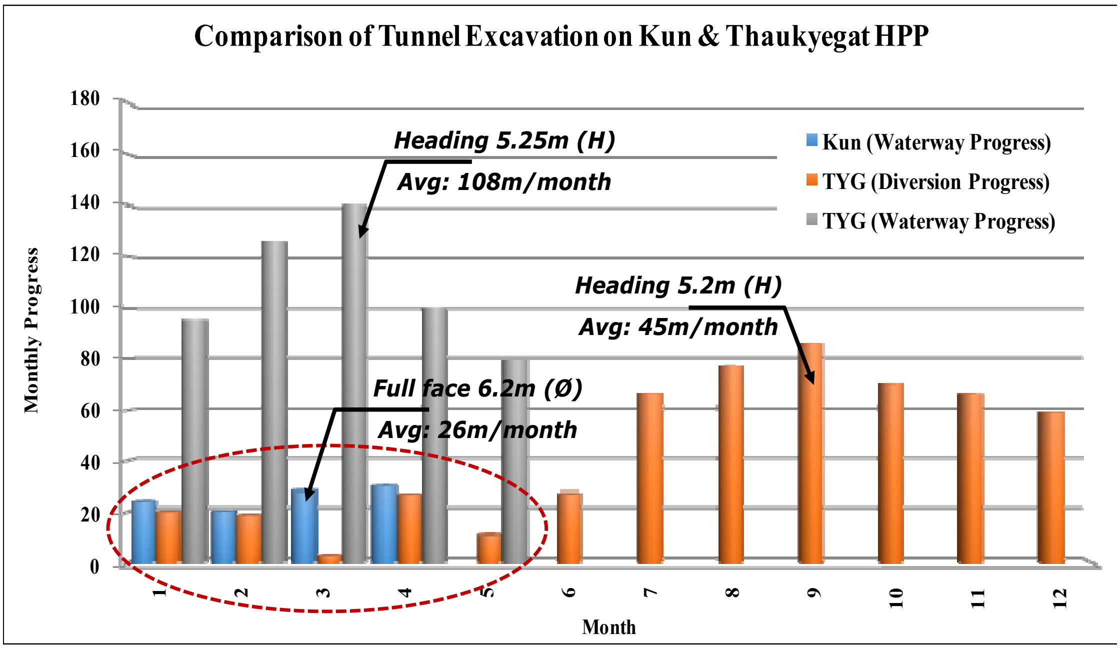


Figure 4.45: Comparison on Tunneling Progress of Kun and Thaukyegat

4.5 Summary

In this chapter, the progress of tunnel excavation on four projects and the focus study on geological assessment of tunneling for two projects were presented. Also, the process of tunnel construction, and cost and schedule risks were compared concerning with unforeseeable geological conditions of the tunnel structure were discussed. The results of comparison of the four projects could be summarized as follows:

Kun Project

According to original economic result, benefit cost ratio (B/C) is 1.31 and Economic Internal Rate of Return (EIRR) is 15.28 % which exceeds the Accounting Rate of Interest normally adopted as 10% to 12%. So the project is economically feasible and it can be allowed for increasing of construction cost up to 25%. But finally, project delay 5 years and construction cost over 72% by serious geological defects on the tunneling and other hydraulic structures. The major defects on tunnel collapse cases are material supply and deficiency of machinery. Project delaying is caused by not only tunnel failure cases but also other structures geological defect and budget delaying during under construction.

Nancho Project

Project delay 4 years because of budget and logistic support delaying, and project cost also overrun about 45%. Project performance and construction management is normal but poor preparation for materials, machinery equipment and budget allocation are seriously defect on project cost and construction schedule.

Thaukyegat Project

Project was postponed one and half year because of delaying of diversion tunnel completion and project cost also just overrun 6% by serious geological defects on tunneling and others structures. Project performance and construction management is going well and better preparation for materials and machinery equipment are leading to project success. Even though some tunnel collapse cases are occurred, tunnel construction is not much effected for whole project. According to closely supervision, fully supply materials and machinery equipment, project cost can keep within the budget even though facing with many geological defects on hydraulic structures.

Paunglaung Project

According to original economic result, benefit cost ratio (B/C) is 1.24 and Economic Internal Rate of Return (EIRR) is 12.1 % and Financial Internal Rate of Return (FIRR) is 30.9 %. So the project is economically feasible and profitable for investment. Although project delay two and half years because of delaying of dam embankment works, project was successfully finished and construction cost can keep within the budget. Project performance and construction management is excellent and perfect preparation for materials and machinery equipment on tunneling works.

CHAPTER 5

GEO-RISK MANAGEMENT ON TUNNELING OF HYDROPOWER PROJECTS IN MYANMAR

5.1 Introduction

Generally, it is well-known that the tunnel failure caused by unforeseeable geological conditions and mismanagement on human which is heavily impact on cost overrunning and schedule delaying of the project. On the other hand, tunnel failure cases can lead to the project fail or success by means of mechanical factor and human factor on the tunneling of hydropower project. In order to mitigate the geo-risk on underground structure, the typical measures can be classified into two types; first is establishment of capacity building for better organizational management, and the other is to carry out the improvement of geological investigation and evaluation of rock mass classification.

Therefore, the success of tunneling on hydropower projects heavily rely on organizational management, and exploration of mountain geology and interpretation of rock mass classification. In addition, the previous result in Chapter 4 indicated that the geo-risk is heavily effect on tunneling and caused the cost overrunning and schedule delaying of the project but project success is mainly depend on construction management. From such a viewpoint, this study aims to identify risk factors related to human factor and mechanical factor concerning with the tunneling of hydropower project through geo-risk management which comprises of “Risk Identification”, “Risk Classification,”, “Risk Assessment” and “Risk Response”.

5.2 Risk Identification on Tunneling of Hydropower Projects

5.2.1 Organizational Responsibility and Accountability of Department

In this section, the authority and responsibility of Department of Hydropower Implementation was discussed to be clearer the circumstances and role of organization in the Ministry of Electric Power. At present in Myanmar, all of the Ministries were formally followed by means of limitation of authority and central finance control system from the central government. As for Ministry of Electric Power also applied the central control system on authority and financial process for the implementation of hydropower projects. Reason why, the financial and decision making process takes minimum two weeks to one month or

more in hierarchy structure of organization as illustrated in Figure 5.1. This process severely defected on tunneling of hydropower projects where were encountering unforeseeable geological problems during under construction, and caused delaying budget which makes difficulty for emergency countermeasure for geo-risk on tunneling. Table 5.1 shows the limitation of authority, accountability and decision making process of the Departmental procedure of Ministry of Electric Power (Aung Myo Hein, 2009).

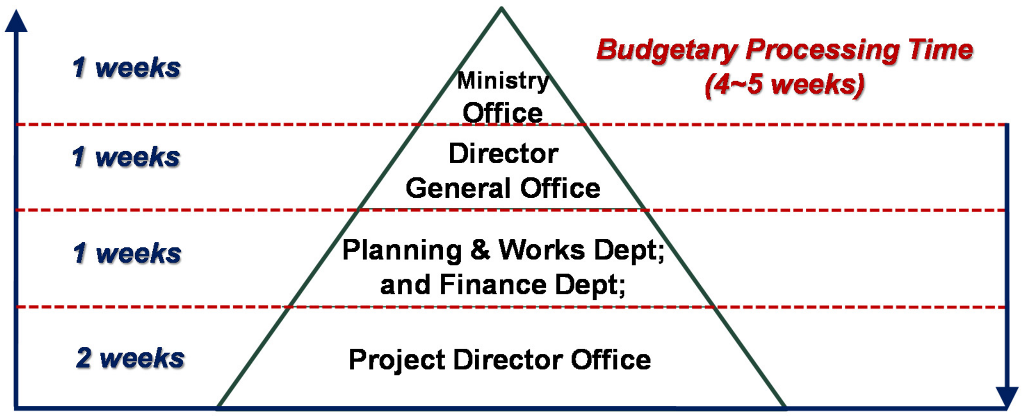


Figure 5.1: Decision Making Process for Additional Activities Cost

Table 5.1: Organizational Responsibility and Accountability of DHPI

	Role at Head Office	Role at Project	Authority & Responsibility	Accountability
Department Level	Director General ⇩ Deputy Director General	Chief Engineer ⇩ Deputy Chief Engineer	❖ *Making Decision and Financial Responsibility for all Projects such as Planning, Design, Finance & Construction.*	• Director General / Chief Engineer *(10 Million Kyats or 1 Million Yen)*
Branch / Project Level	Director ⇩ Deputy Director	Supreme Engineer ⇩ Assistant Supreme Engg;	❖ *Planning of Scope* ❖ *Schedule Approve* ❖ *Design Approve* ❖ *Design Variation* ❖ *Negotiation with different Parties* ❖ *Change of Requests* ❖ *Procurement*	• Director / Supreme Engineer *(3 Million Kyats or 0.3 Million Yen)*
Functional Level	Assistant Director	Executive Engineer	❖ *Planning of Works (Time, Scope, Resource)* ❖ *Managing Project Team* ❖ *Achievement of Project Works* ❖ *Financial Control* ❖ *Method of Construction Works* ❖ *Dispute Resolutions within Project Teams*	• Assistant Director / Executive Engineer *(500,000 Kyats or 50,000 Yen)*
Operation Level	Staff Officer	Assistant Engineer	❖ *Program of Works & Schedule* ❖ *Construction Works* ❖ *Task Performing* ❖ *Progress Report* ❖ *Feedback* ❖ *Local Sub Contractors Payment Control*	• Staff Officer / Assistant Engineer *(100,000 Kyats or 10,000 Yen)*

*** For more than 10 Million Kyats or 1 Million Yen, Ministry Office permission is required for financial authority.*

Consequently, additional cost for extra works or extra activities by means of unforeseeable geological problems in the tunneling works is usually delayed due to central cost control system. On the other hand, delaying countermeasure for geo-risk failure on tunneling may result the higher potential of tunnel failure risks and increasing of repair cost. Therefore, it is necessary for organization to upgrade project management technology and enhance the capability of organization by structuring the capacity building of hydropower engineering**.**

5.2.2 Risk Identification and Evaluation on Tunneling

In this part, risk factors on tunneling of hydropower projects were presented and evaluated. Based on the geo-risk of tunneling, some key risk factors are identified by means of human factor and mechanical factor. Four key risks which is highly impact to the efficiency of tunneling performance are organization, procurement, finance and construction in the hydropower development.

In the organization factor, there is three potential risks such as technical constraints, lack of skilled workforce and human mistake. The evaluation should be done as follows:

- Technical constraints should be improved well by means of preparing human resource development.
- Lack of skill workforce should be managed well by allocating right person and enough capacity for the project site.
- Human mistake should be avoided well by organizing and making right decision for the project.

In the procurement factor, there is two potential risks such as insufficient major equipment and resources constraint. The evaluation should be done as follows:

- Insufficient major equipment should be prepared well by providing required machinery equipment to be enough for each hydropower tunneling.
- Resources constraint should be managed well by preparing resources ahead before starting the tunneling works.

In the finance factor, there is two potential risks such as budget delay and budget insufficient. The evaluation should be done as follows:

- Budget delay should be avoided well because in term of finance, delaying of budget is a risk factor which may affect the tunneling on serious geological area.

- Budget insufficient should be supplied well because of well preparation of material and machinery is mainly depend on availability of budget, but insufficient of budget may defect on tunneling works of weak geological area.

In the construction factor, there is three potential risks such as unforeseen ground condition, geological observation, and poor working condition which is mainly associated with mechanical factors. The evaluation should be done as follows:

- Unforeseen ground condition should be investigated well by means of geological investigations such as drilling, seismic, resistivity etc.
- Geological observation should be evaluated well by providing well observation and proper evaluation which can be minimize the geo-risk and cost effective on tunneling.
- Poor working condition should be avoided well by improving well preparation and frequently discussion on job site.

The following table 5.2 presents the key risks factors and risk evaluation on tunneling of hydropower projects.

Table 5.2: Potential Risks and Evaluations on Tunneling of Hydropower Projects

	Potential Risks	Evaluations
Organization	▪ **Technical Constraints** should be improved well. ▪ **Lack of skilled workforce** should be managed well. ▪ **Human mistake** should be avoided well.	▪ To prepare **human resource development.** ▪ To allocate **right person and enough capacity** for the project site. ▪ To **organize and right decision** for the project.
Procurement	▪ **Insufficient major equipment** should be prepared well. ▪ **Resources constraint** should be managed well.	▪ Required machinery equipment should be enough for each **Hydropower Tunneling**. ▪ To prepare resources ahead before starting the **Tunneling Works.**
Finance	▪ **Budget delay** should be avoided well. ▪ **Budget insufficient** should be supplied well.	▪ In the term of finance, **delaying of budget** is a risk factor which may affect the tunneling on serious geological area. ▪ Well preparation of material and machinery is mainly depend on availability of budget, but insufficient of budget may defect on **Tunneling Works**.
Construction	▪ **Unforeseen Ground Condition** should be investigated well. ▪ **Systematic Geological Observation** should be evaluated well. ▪ **Poor Working Condition** should be improved well.	▪ It can be investigated well by geological investigations such as **drilling, seismic, resistivity etc.** ▪ Well **observation and evaluation** can minimize the geo-risk and cost effective on tunneling. ▪ To improve poor working condition, **discussion and well preparation** on job site is essential.

In this section, the paper identifies that geo-risk factors involved in tunnel construction are mainly divided into two parts: "geological condition" and "construction management system", which are perceived as "Natural Hazard" and "Man-made Hazard", respectively.

5.3 Risk Classification on Tunneling of Hydropower Projects

5.3.1 Evaluation on Cost and Schedule of Sittaung Hydropower Projects

Based on previous study in Chapter 4, Table 5.3 and 5.4 present the comparison of evaluation on cost and schedule risk by geo-risk on tunneling of seven hydropower projects from Sittaung valley. Almost projects are facing with cost over running and schedule delaying, and some projects are heavily impact on project success by means of cost and schedule. To overcome cost and schedule risks for hydropower development, systematic geological observation, procurement of tunnel tools and equipment are essential for tunnel excavation on every kind of geological conditions.

Table 5.3: Comparison on Projects Scale and Geological Conditions of the Seven Projects

No.	Projects	Power		Geology Condition	Remarks
		Installed (MW)	Energy (Gwh)		
1	Kun	60	190	*Metasandstone & Mudstone (weak)*	*By MOEP Con; Period: 2002~2012*
2	Phyu	40	120	*Metasandstone & Mudstone (weak)*	*By MOEP Con; Period: 2002~2014*
3	Kabaung	30	120	*Metasandstone & Mudstone (weak)*	*By MOEP Con; Period: 2003~2008*
4	Yenwe	25	123	*Metasandstone & Mudstone (weak)*	*By MOEP Con; Period: 2001~2007*
5	Thaukyegat	120	605	*Phyllite, Schist & Metasandstone (weak)*	*By Local Company Con; Period: 2008~2013*
6	Nancho	40	152	*Granite & Granitic Gneiss (good)*	*By MOEP Con; Period: 2005~2014*
7	Paunglaung	280	911	*Granite & Granitic Gneiss (good)*	*By MOEP Con; Period: 1997~2005*

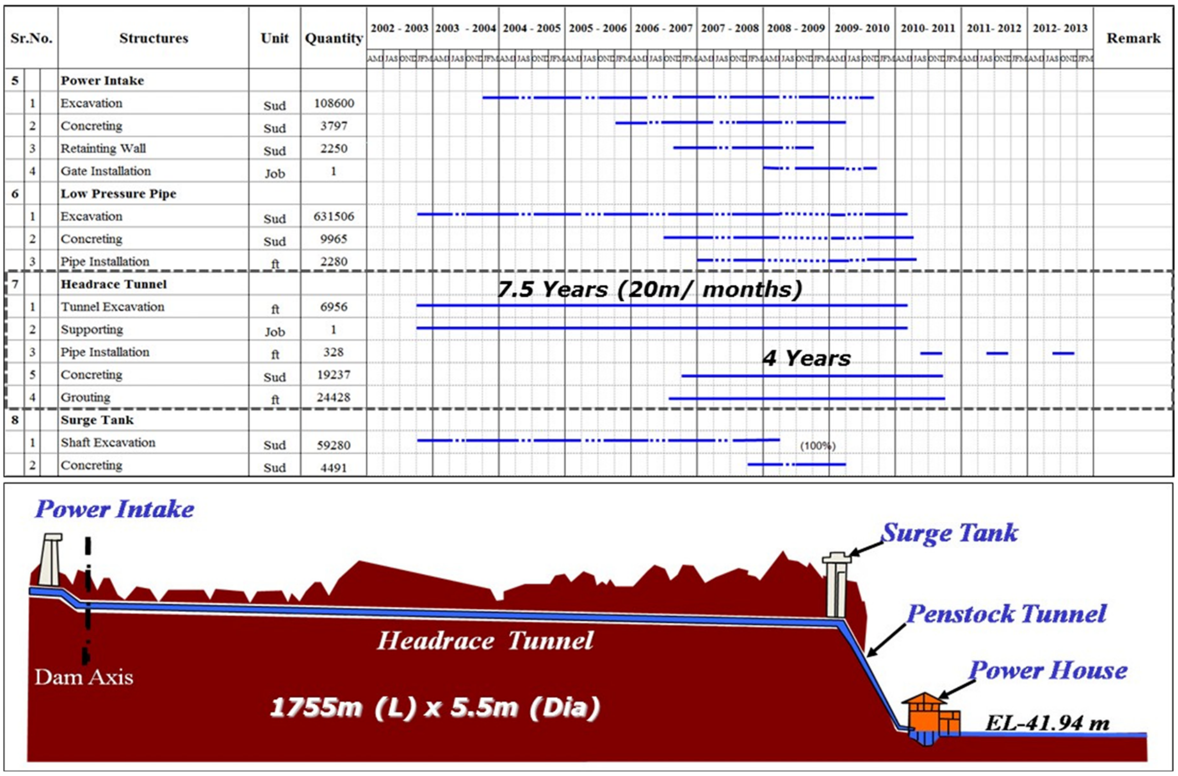
Table 5.4: Comparison on Cost and Schedule of the Seven Projects

No.	Projects	Construction Period (years)			Project Cost (MUSD)			Remarks
		Planning	Actual	Delay	Initial	Final	Over (%)	
1	Kun	6	11	**5**	63.22	108.45	**72**	***1MWcost is 1.8 M$***
2	Phyu	5	12	**7**	24.30	28.23	**16**	***Only Power Portion***
3	Kabaung	4	5	**1**	13.30	16.31	**23**	***Only Power Portion***
4	Yenwe	4	6	**2**	12.96	14.53	**12**	***Only Power Portion***
5	Thaukyegat	3.5	5	**1.5**	234.94	249.62	**6**	***1MWcost is 2.1 M$***
6	Nancho	5	9	**4**	41.31	60.10	**45**	***1MWcost is 1.5 M$***
7	Paunglaung	5.5	8	**2.5**	369.67	371.16	**0.4**	***1MWcost is 1.3 M$***

5.3.2 Cost and Schedule Risk on Tunneling of Kun Project and Thaukyegat Project

The uncertainty of geo-risk is highly impact on tunnel construction of hydropower projects. According to comparactive study of two projects from Sittaung valley, it can be seen that underground works are one of the greatest sources of cost and schedule risk for hydropower projects.

For the Kun project, there are two main structures such as main dam and waterway tunnel which are critical on construction schedule for the project. Initially project was planned to be finished within 6 years but, it was finally take time for 11 years from 2002 to 2012. One of the reasons for construction delaying was effect by complex geological condition along the waterway tunnel, and faced frequently difficulty of excavation works and failure events on tunneling works. Even though plan for tunneling works was 3.5 years initially, over all construction period was 8 years which is associated excavation for 7.5 years and concrete lining for 4 years as shown in Figure 5.2.



Sr.No.		Structures	Unit	Quantity	Remark
5		Power Intake			
	1	Excavation	Sud	108600	
	2	Concreting	Sud	3797	
	3	Retaining Wall	Sud	2250	
	4	Gate Installation	Job	1	
6		Low Pressure Pipe			
	1	Excavation	Sud	631506	
	2	Concreting	Sud	9965	
	3	Pipe Installation	ft	2280	
7		Headrace Tunnel			
	1	Tunnel Excavation	ft	6956	
	2	Supporting	Job	1	
	3	Pipe Installation	ft	328	
	5	Concreting	Sud	19237	
	4	Grouting	ft	24428	
8		Surge Tank			
	1	Shaft Excavation	Sud	59280	
	2	Concreting	Sud	4491	

Figure 5.2: Construction Schedule of Kun Project (Source: DHPI, 2013)

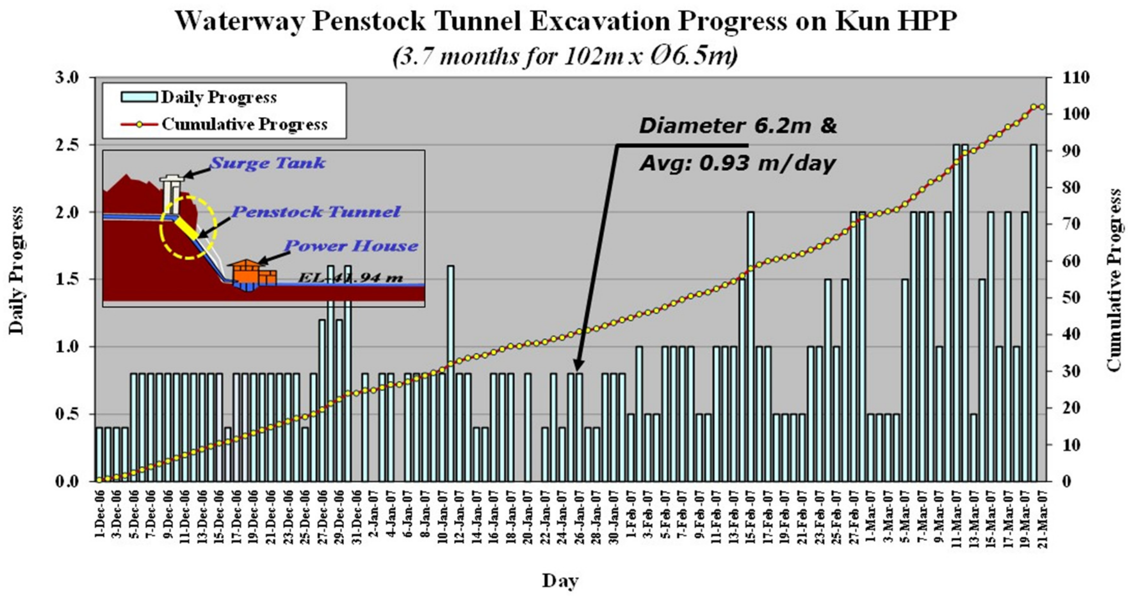


Figure 5.3: Excavation Progress on Penstock Tunnel of Kun Project

One of the serious geological portions, Penstock tunnel which is located in the fractured zone between the surge tank and outlet portal, cannot speedy the excavation works and auxiliary tunneling method such as fore-poling, ring-cut and rock bolt combined with the conventional method are applied to make sure tunneling against poor geology. Penstock tunnel excavation was take time 3.7 months for 102 m long and monthly progress is 26 m under the condition

of sheared mudstone zone as illustrated in Figure 5.3. According to serious geological defects on tunnel alignment, the waterway tunnel initial estimated cost is 8.5 MUSD but final cost is about two times, 16.8 MUSD which is one of the reasons for cost overrun and schedule delay on the project as tabulated in Table 5.5 and 5.6.

For the Thaukyegat project, there are two tunnel structures such as diversion tunnel and waterway tunnel which are important sequence for the construction project. Initially project was planned to be finished within 3.5 years which is actually tight schedule for the large scale hydropower project, but it was finally take time for 5 years from 2008 to 2013. Diversion tunnel works were planned for 16 months, but overall construction period for diversion tunnel was take time for 26 months which is associated excavation for 18 months and concrete lining for 8 months because of geological defects where are facing highly weathered rock with ground water around the inlet and outlet portal. For the waterway tunnel works were also planned for 16 months, but overall construction period for waterway tunnel was 21 months which is associated excavation for 8 months and concrete lining for 11 months. Total cost for waterway tunnel is within the budget as shown in Table 5.5 and 5.6. Project performance and construction management is going well by perfect preparation for materials and machinery equipment on construction works of Thaukyegat project. The schedule and progress of both tunnels are demonstrated in following figures.

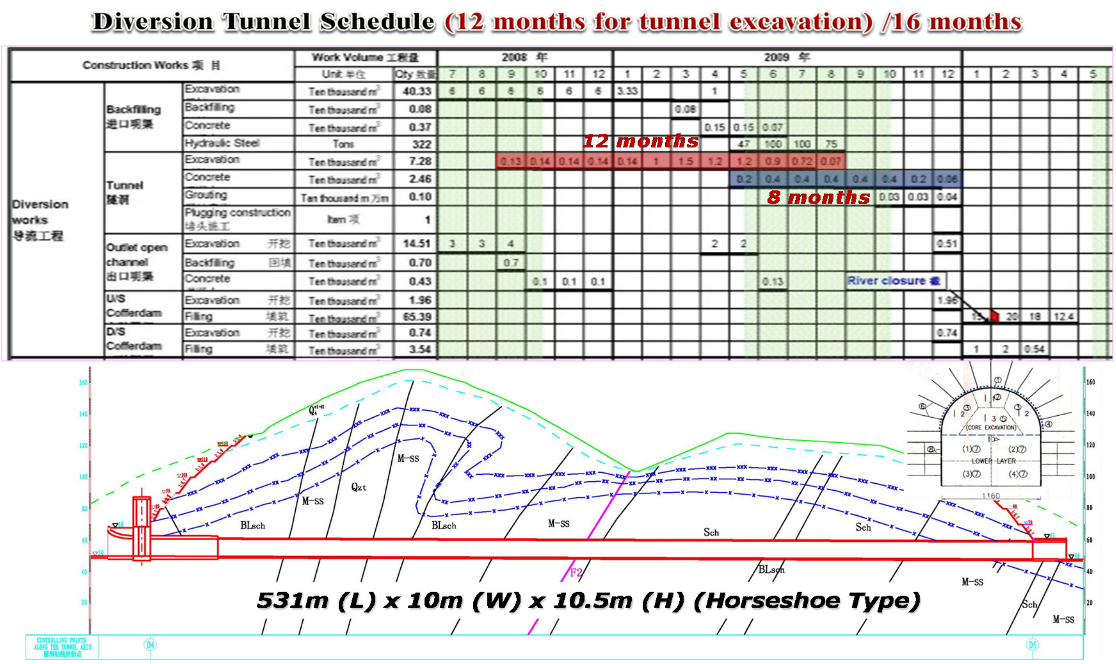


Figure 5.4: Diversion Tunnel Construction Schedule of Thaukyegat Project

(Source: Wan Kyi, 2009)

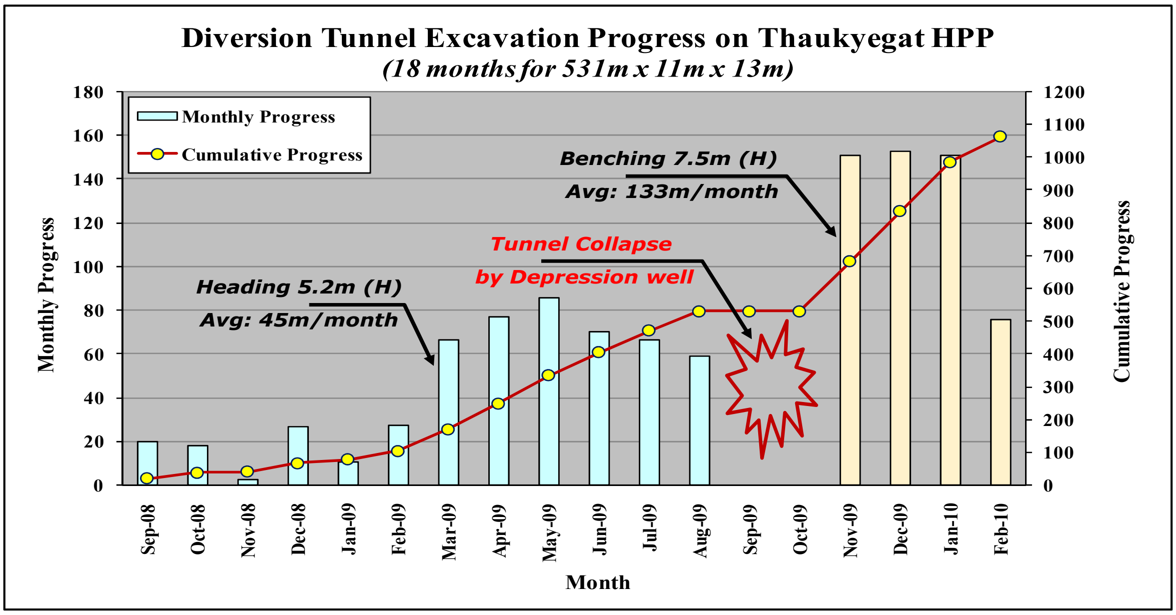


Figure 5.5: Diversion Tunnel Excavation Progress of Thaukyegat Project

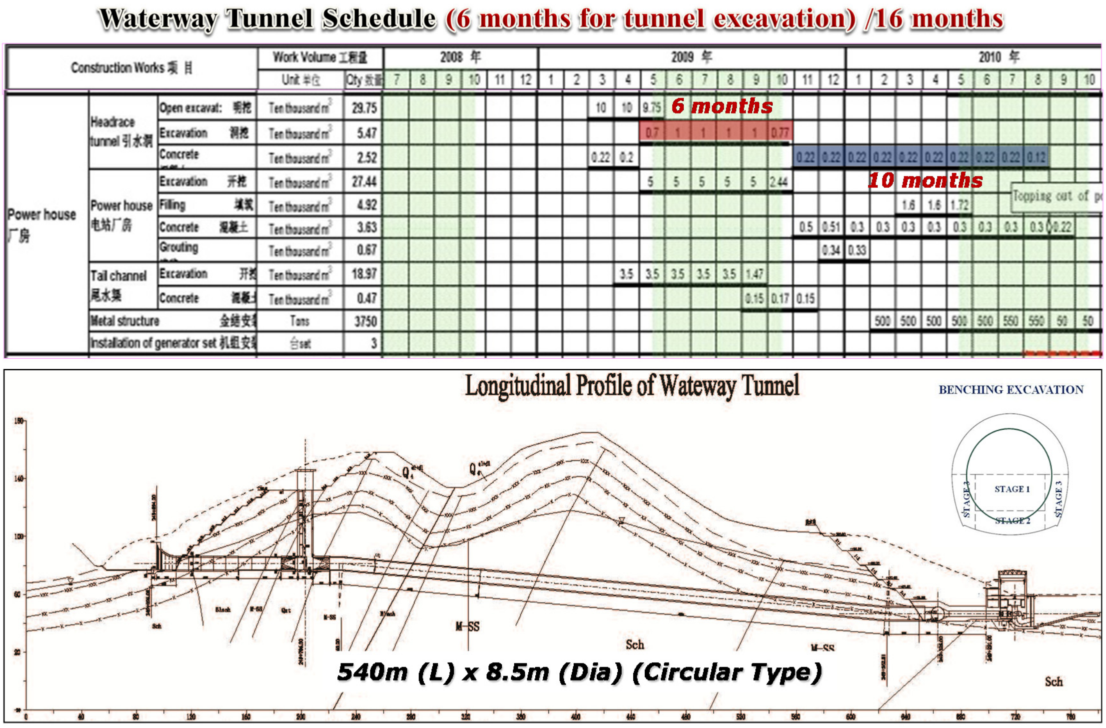


Figure 5.6: Waterway Tunnel Construction Schedule of Thaukyegat Project

(Source: Wan Kyi, 2009)

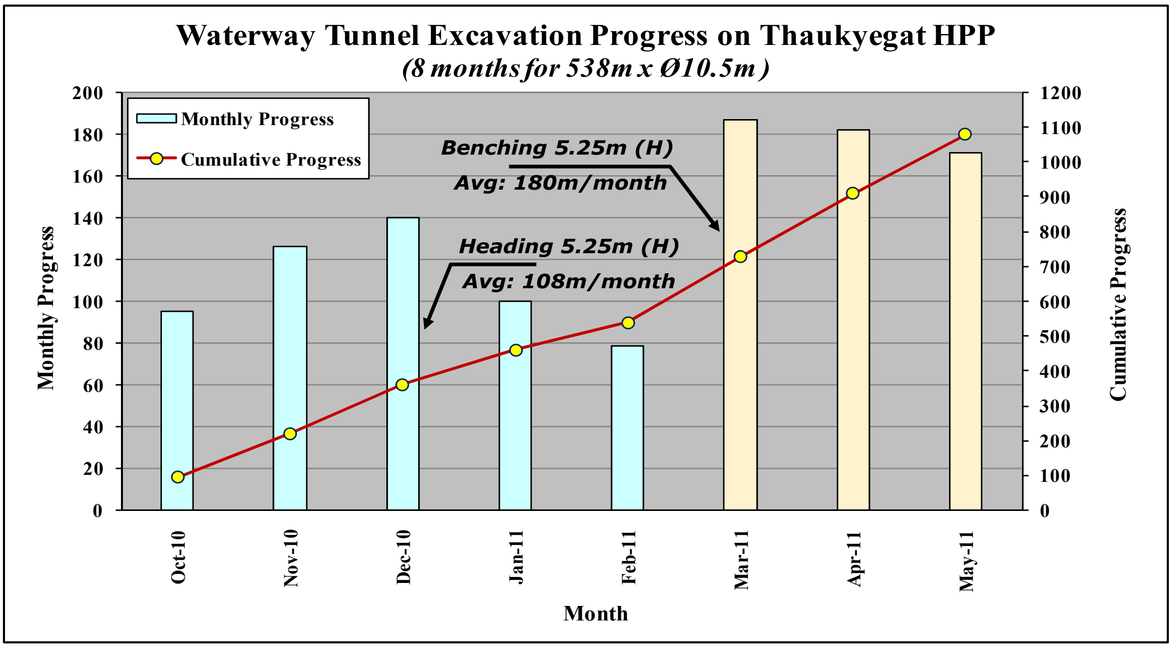


Figure 5.7: Waterway Tunnel Excavation Progress of Thaukyegat Project

Table 5.5: Comparison on Projects Scale and Construction Period of Tunneling on Kun and Thaukyegat

No.	Projects	Power		Construction Period	Remarks
		Installed (MW)	Energy (Gwh)		
1	Kun	60	190	*Plan: 2002~2007 (6 years)* *Actual: 2002~2012 (11 years)*	*By MOEP*
2	Thaukyegat	120	605	*Plan: 2008~2011 (3.5 years)* *Actual: 2008~2013 (5 years)*	*By Local Company*

Table 5.6: Comparison on Cost and Schedule of Tunneling on Kun and Thaukyegat

No.	Projects	Tunnel	Tunneling Period (Years)			Tunneling Cost (MUSD)			Remarks
			Plan	Actual	Delay	Initial	Final	Over (%)	
1	Kun	Waterway (1755mx5.5m)	3.5	7.5	4	8.49	16.75	49	*Material & Labour Cost*
2	Thaukyegat	Diversion (531mx10m)	1	1.5	0.5	-	-	-	
		Waterway (538mx8.5m)	0.5	0.67	0.17	12.46	12.46	0	*Material & Labour Cost*

In this section, it is highlight that classification of geological condition whether it is poor or good is a key factor to identify geo-risk factor, which impact cost overrun/ delay of construction period on tunneling of hydropower project.

5.4 Risk Assessment on Tunneling of Hydropower Projects

According to detailed risk assessment study in Chapter 4, in the first pair of comparison on tunneling of Kun project and Nancho project, geological conditions on both projects are totally different which geo-risk on tunneling of Kun project is very high but less impact on Nancho project. For Kun projects, tunnel excavation cannot be speedily preceded under the situation of very poor geological conditions and unforeseen condition of the underground works. However, for Nancho project, tunnel excavation can speedy without any failure cases almost of waterway tunnel. Anyhow, both projects are facing with cost overrunning and construction delaying as tabulated in Table 5.7. In the comparison table, it mainly focus on geological factor (mechanical factor) and organizational management (human factor) which is concerning on cost overrunning and construction delaying of the projects.

Table 5.7: Review on Tunneling of Kun Project and Nancho Project Case Study

Situation	KUN	Nancho
1. Geological Condition		
(1) Lithology	Sandstone, Mudstone (weak)	Granite, Granitic Gneiss (good)
2. Construction Achievement		
(1) Completion Target	5 year Delay	4 year Delay
3. Project Cost		
(1) Budget	72% Over Run (Over all Project Cost)	45% Over Run (Over all Project Cost)
4. Organization Condition		
(1) Manage: & Super:	Good	Good
(2) Work Plan	Normal	Normal
(3) Cooperation	Good	Good
(4) Skill of Workers	Normal	Normal
(5) Financial Support	< Normal	< Normal
(6) Logistic Support	< Normal	< Normal

In the second pair of comparison on tunneling of Thaukyegat project and Paunglaung project, geological conditions on both projects are totally different which geo-risk on tunneling of Thaukyegat project is very high but less impact on Paunglaung project. For Thaukyegat projects, tunnel excavation was not well preceded associated with some tunnel failure cases under the situation of very poor geological conditions and unforeseen condition of the underground works. In the same manner, tunnel excavation of Paunglaung project was speedily preceded without any failure cases because of excellent geological conditions along the tunneling works. Finally, both projects are successfully finished even though small amount of cost overrunning and construction delaying as tabulated in Table 5.8. In the comparison table, it mainly focus on geological factor (mechanical factor) and organizational management (human factor) which is mainly concerning on cost overrunning and construction delaying of the projects.

Table 5.8: Review on Tunneling of Thaukyegat Project and Paunglaung Project Case Study

Situation	Thaukyegat	Paunglaung
1. Geological Condition		
(1) Lithology	Phyllite, Schist, Meta-sandstone (weak)	Granite, Granitic Gneiss (good)
2. Construction Achievement		
(1) Completion Target	1.5 year Delay	2.5 year Delay
3. Project Cost		
(1) Budget	6% Over Run (Over all Project Cost)	within the Budget (Over all Project Cost)
4. Organization Condition		
(1) Manage: & Super:	Good	Good
(2) Work Plan	Normal	Good
(3) Cooperation	Good	Excellent
(4) Skill of Workers	Normal	Good
(5) Financial Support	Good	Good
(6) Logistic Support	Good	Excellent

For those risk assessment on four projects, it can be demonstrated as Figure 5.8 to make clear for the classification of risk factor and construction fail or success concerning with the mechanical factor and human factor on tunneling of the hydropower projects.

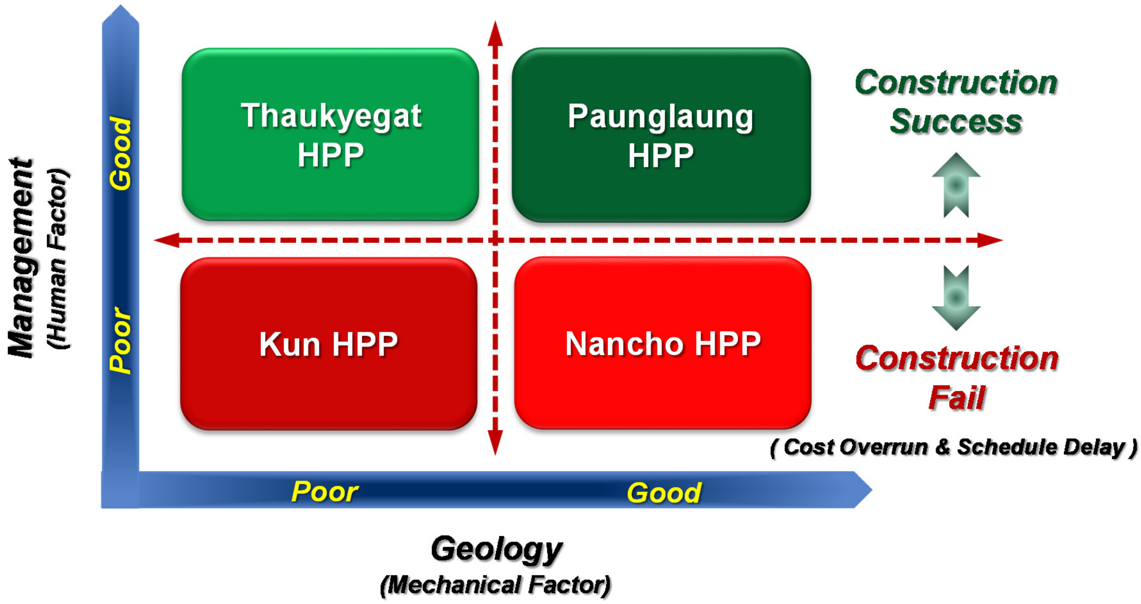


Figure 5.8: Review on Tunneling of Case Study of Four Projects

In this section, the paper shows case studies for four tunnels associated with development of hydropower projects in Myanmar, focusing on cost overrun and delay of construction period. In the case of tunnels constructed in good geological condition, while construction management system does not matter from a view point of tunnel stability during construction, it may lead to cost overrun/ delay of construction period. On the other hand, impropriate prediction of "geological condition" causes significant cost overrun and delay of construction period, and "poor construction management system" increases additional losses.

5.5 Risk Response on Tunneling of Hydropower Projects

Based on case study results, it would be recommended that the development of tunneling in hydropower projects, the most important is strengthening on "poor construction management system" human factors and "poor geological condition" mechanical factors of tunneling practices.

In order to scope with difficulties associated "poor construction management system" in Myanmar, following remedial measure would be expected.

- Skill of construction works.
- Decision-making system in response to realization of unexpected poor geological condition.

- Procurement system.
- Financial system.

In order to scope with difficulties associated "poor geological condition", following remedial measure would be expected.

- Improvement of underground geological investigation (Exploratory Drilling, Geophysical Survey and Rock Mechanics Testing, etc.).
- Evaluation on rock mass classification along the tunnel by using proper forecasting methodology (Core Point Method, Indicator Krinking or Neutral Networks, etc.).
- Establishment of database system on tunnel specifications and method of statement based on past hydropower tunnels data in Myanmar.

5.5.1 Improvement of Risk Control on Human Factor

At present, large scale hydropower development is continuously increased from last two decades to until now in Myanmar. So the demand of experience engineers and workforce becomes higher in hydropower constructions, and facing with shortage in the construction organizations. Insufficient of technical competency and skilled workforce is causing possession of risk in hydropower construction engineering. Therefore, competency of skilled workforce is required for the long-term organization strategy by providing required training to employees and awareness of hydropower engineering also have to improve for the minimization of construction risks.

At the same time, it is important to establish a capacity building for hydropower engineers and skilled labors. The most efficient way of structuring for capacity building is education in technical colleges and universities which plays an important role to increase the skilled engineers. It can promote institutions in construction engineering of hydropower projects. Research capacity building is essential for skilled workforce, technical competency and better construction management. The purpose of human resources development and research capacity building is strengthening professional, organizational and management capacities of hydropower development.

However, departmental procedure of Ministry of Electric Power is followed by the governmental rules and regulations, it can be improved by means of technical work force

and capacity building of human resources for the future hydropower development. As capacity building of human resources combines engineering principles with technical knowhow and working experience of tunneling works, it requires effective teamwork as essential which is consisting of designers, construction engineers, mechanical operators, geologists and skilled workforce.

5.5.2 Improvement of Risk Control on Mechanical Factor

On the other hand, geo-risk reduction on tunneling of hydropower projects also can be secure by improving of mechanical factor. When carrying out preliminary design for tunnels discrete data such as rock mass classifications are used as mechanical parameters derived from the results of boring surveys. Then, such discrete data are interpret to correspondence tunnel alignment with certain numerical referential values such as RMR or Q System, it is possible to predict the physical property values of target tunnel alignment by geologist. It can be produced predictive data for applying the geological assessment on tunneling of hydropower projects. As aforesaid, the comparison of predicted results with actual Q System of waterway tunnel of Thaukyegat project can highlight the less risk on cost and schedule of tunneling.

For mitigation of geo-risk on tunneling, it is necessary to improve the geological investigation such as exploratory drilling and geophysical survey (seismic refraction, electrical resistivity, etc.), and geostatistics evaluation by using suitable forecasting methodology such as Core Point Method, Indicator Krinking or Neutral Networks as demonstrated in Figure 5.9. From the view point of mechanical factor, in fact most tunnel collapses where is mostly applied New Austrian Tunneling Method (NATM) in the tunneling of Sittaung hydropower projects are connected with unexpected ground conditions and insufficient data to work with. NATM is a safe method if properly applied which is approved by means of Sittaung hydropower projects and it requires such as theory and evaluations, experience of engineers and higher performance of skilled labors.

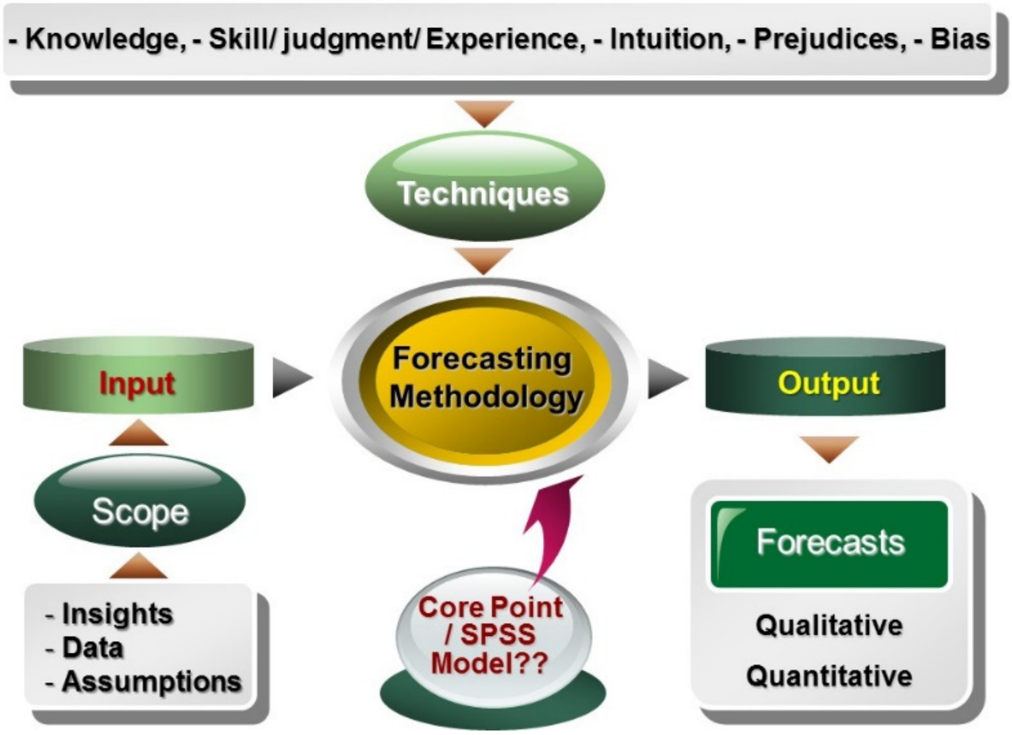


Figure 5.9: Forecasting Process of Geostatistics Evaluation on Mountain Tunneling

(Source: Flanagan & Norman, 1993)

5.6 Impression on Tunneling Practices of Japan

There is some differences on tunneling methodology of investigation between Japan and Myanmar. In Myanmar, the following investigation procedures of mountain tunneling are carried out through feasibility stage to construction stage.

- Site reconnaissance survey by geologist (photogeological study, ground investigation, etc.)
- Ground conditions (topography, geology, ground water, etc.)
- Exploratory drilling (portals, faults, gully area, etc.)
- Preparing geological plan and profile of the tunnel

In Japan, the following investigation procedures of mountain tunneling are carried out through feasibility stage to construction stage.

- Site reconnaissance survey by Geologist (photogeological study, ground investigation, etc.)

- Ground conditions (topography, geology, ground water, etc.)
- Exploratory drilling, geophysical survey (seismic refraction, electrical resistivity, etc.) and rock mechanics testing, etc.
- Evaluation on rock mass classification along the tunnel
- Preparing geological plan and profile of the tunnel

As above mentioned, it is clearly seen that Japanese tunneling investigation process is very detail and take time to clarify the unforeseeable geological conditions and evaluation of mountain geology before starting the construction. It is noticed that during construction also special take care on tunnel excavation procedure by applying NATM method and well preparation on construction materials, heavy equipment, safety procedure and environmental conservation which is getting knowledge from the site visit of high way tunnel construction from Oaza Saho, Ibaraki City to Aomatani, Minoh City in Osaka prefecture. Tunneling procedure of highway tunnel construction by Taisei Corporation which is sub-contractor from west branch of West Nippon Expressway Company Limited and site visit recorded photos are presented in Appendix E. One of the most impressions is data base system on specifications and tunnel construction methods which is based on past records and experiences of tunneling practices in Japan.

5.7 Summary

In this chapter, geo-risk management process which comprises of “Risk Identification”, “Risk Classification,”, “Risk Assessment” and “Risk Response” are presented. The seven hydropower projects are selected for comparison of cost and schedule, and two projects are selected for focus study on geological failure behavior of the tunnel structures where are located in complex geological area. Another comparative study was also set for four projects which are having similar tunnel structures and located in different geological conditions. The findings were obtained by comparing of the geo-risk on tunneling of hydropower projects and could be summarized as follows:

- Description of risk identification by organizational system and potential risks of hydropower tunneling.

- Configuration of risk classification by defining of geological condition on mountain tunneling.
- Description of risk assessment by comparative study of four tunnels associated with development of hydropower projects.
- Clarification on risk responses by human factor and mechanical factor which is mainly impact on tunnels construction of hydropower projects.
- Highlight on impression of Japanese tunneling practices.

Based on above study, it is noticed that tunneling in the region of good geology are simple and poor construction does not much effect on tunneling, but tunnel construction in poor geology face much complicated disturbances leading to collapse and poor construction also heavily effect on tunneling of hydropower project.

Consequently, human factor and mechanical factor are essential for geo risk reductions which can improve the organizational management, advancing of geological investigation and estimation of rock mass on weak geological area. It can be concluded that the most geo-risk behaviors on weak geology among the implementation of hydropower projects are not only depend on poor geology but also mismanagement on human factors.

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

In Myanmar, to consider future strategies of institutions in the tunnel construction of hydropower projects, geo-risk management plays a vital role to complete the projects within the budget allocation and on time as targeted. According to risk classification, there are many geo-risk factors in the hydropower projects. However, in this study, unforeseeable geo-risk on tunneling is chosen to figure out risk factors and their responses which are heavily impact on project cost and construction schedule of Sittaung hydropower projects. The aim of this study is understanding on present situation of tunneling methods and development of tunneling technology by identifying and responding of risk based on cost overrunning and schedule delaying of hydropower projects. The study purpose is summarized as follow:

- To clear that geo-risk dominantly impacts on project cost and construction schedule. Especially, weak geological area should be paid more attention on tunneling.
- To point out that most of projects on the weak geological area are faced with different types of tunnel failure mechanism which may impact on construction schedule and project cost.
- To figure out the pushing on construction progresses without much taking care on situation of work sites which is facing with serious geological problems.
- To find out the geo-risk impacts on tunneling relative to hydropower development.
- To identify that geo-risk management will not only secure the financial and schedule burden of the project but also bring good practices of preserving and enhancing project quality and social well-beings in hydropower development.

In the Chapter 4, for the geo-risk assessment, some projects case study are carried out which is the progress of tunnel excavation on four projects and the focus study on geological assessment of tunneling for two projects. And also, the process of tunnel construction, and cost and schedule risks were compared and discussed concerning with unforeseeable geological conditions of the tunnel structure.

In the Chapter 5, geo-risk management on tunneling of hydropower projects are presented. The seven hydropower project are selected for comparison of cost and schedule, four projects which are having similar tunnel structures and located in different geological conditions are presented on project performance, and two projects are selected for focus study on geological failure behavior of the tunnel structures, cost and schedule risks. The findings were obtained from comparing of the geo-risk, and risk factors on tunneling of hydropower projects.

In the geo-risk management study process, the composition of research phases are "Risk Identification", "Risk Classification,", "Risk Assessment" and "Risk Response" by means of case study on Sittaung Valley Hydropower Projects.

a) Risk Identification on Tunneling

First, the paper identifies that geo-risk factors involved in tunnel construction are mainly divided into two parts: "geological condition" and "construction management system", which are perceived as "Natural Hazard" and "Man-made Hazard", respectively. The verification of risk factors identification is important for the study because the rationale of organizational management and mechanical geo-risk factor to tunneling process is necessary to find out for the cases study in order to gather the concrete results of the analysis.

b) Risk Classification on Tunneling

As for "geological condition", the paper suggests that classification of geological condition whether it is poor or good is a key factor to identify geo-risk factor, which impact cost overrun/ delay of construction period. The objective of risk classification is to figure out the dominant weak points on tunneling of poor or good geological conditions and to control the risks by improving the human and mechanical factors which is majorly impact on tunneling of hydropower projects.

c) Risk Assessment on Tunneling

The paper shows case studies for four tunnels associated with development of hydropower projects in Myanmar, focusing on cost and delay of construction period. As for "geological condition", it points out that there is big difference between rock classifications such as Rock Tunneling Quality Index (Q)/ Rock Mass Rating (RMR) evaluated in prior to excavation and those measured during excavation. At the area where the gap is significant, tunnel collapsed. Risk factors pointed out are follows:

- Limitation of ground investigation data
- Poor evaluation of rock mass
- Poor/ Impropriate construction works
- Poor procurement of tunnel support members
- Lack of finance to procure tunnel support members

In the case of tunnels constructed in good geological condition, while construction management system does not matter from a view point of tunnel stability during construction, it may lead to cost overrun/ delay of construction period. On the other hand, impropriate prediction of "geological condition" causes significant cost overrun and delay of construction period, and "poor construction management system" increases additional losses.

d) Risk Response on Tunneling

Based on case study results, it would be recommended that the development of tunneling in hydropower projects, and the most important is strengthening on human factors and mechanical factors of tunneling practices.

In order to scope with difficulties associated "poor construction management system" in Myanmar, following remedial measure would be expected.

- Skill of construction works.
- Decision-making system in response to realization of unexpected poor geological condition.
- Procurement system.
- Financial system.

In order to scope with difficulties associated "poor geological condition", following remedial measure would be expected.

- Improving underground geological investigation (Exploratory Drilling, Geophysical Survey and Rock Mechanics Testing, etc.).

- Evaluation on rock mass classification along the tunnel by using proper forecasting methodology (Core Point Method, Indicator Krinking or Neutral Networks, etc.).

- Establishment of database system on tunnel specifications and method of statement based on past hydropower tunnels data in Myanmar.

In conclusion, it is approved that the most geo-risk behaviors on weak geology among the implementation of hydropower projects are not only depend on weak geology but also mismanagement on human factors of organizational management. The findings were obtained from comparing of the geo-risk, and risk factors on tunneling of hydropower projects. For the Ministry of Electric Power, the major findings of this study to be improve in terms of geo-risk factors for tunneling of hydropower projects are organizational management, procurement sector, finance sector and construction sector. Finally, the systematic approach adopted for geo-risk management on tunneling can be minimized the cost overrunning and schedule delaying of the hydropower projects.

6.2 Recommendations for Further Study

According to comparative study of tunneling practice, it is noticed that tunneling in the region of good geology are simple and poor construction does not much effect on tunneling, but tunnel construction in poor geology face much complicated disturbances leading to collapse and poor construction also heavily effect on tunneling. Therefore it is essential for geo risk reduction by improving human factors and mechanical factors. For the human factors, it is important to establish a capacity building for hydropower engineers which can promote institutions in construction engineering of hydropower project. Research capacity building is essential for skilled workforce, technical competency and better construction management on hydropower development in future. For the mechanical factors, it is necessary for advancing geological investigation and estimation of rock quality which can promote tunnel excavation and supporting system on weak geological area.

In addition, it is noticed that forecasting methodology is effective on evaluation of mountain geology and preliminary cost estimation of tunnel construction, and tunnel specifications are also well established with database system by referring Japanese practices. Therefore, it is proposed for evaluation on geo-risk reduction by advancing geological investigation such as the seismic refraction or the electrical resistivity, and estimation of rock mass by using proper forecasting methodology which can improve rock mass classification and cost estimation.

At present study, there is some constraint to focus on mechanical geo-risk factors for tunneling because the first reason is limited available geological investigation data in Myanmar and it cannot give proper results for evaluation of rock mass, and the second is the time constraint for study period.

In the Ministry of Electric Power, the research capacity building should be established by enhancing on four sectors which is heavily impacted on tunneling of hydropower projects concerning with human factors and mechanical factors in the future. The prioritization should be considered on the wide range of capacity building for the improvement of tunnel construction performance on each projects.

For further study, it can be extended to focus on geophysical survey and learn on geological evaluation by using proper forecasting methodology to apply in tunneling works of hydropower projects, and establishment of database system on tunnel specifications and method of statement based on past hydropower tunnels data in Myanmar.

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